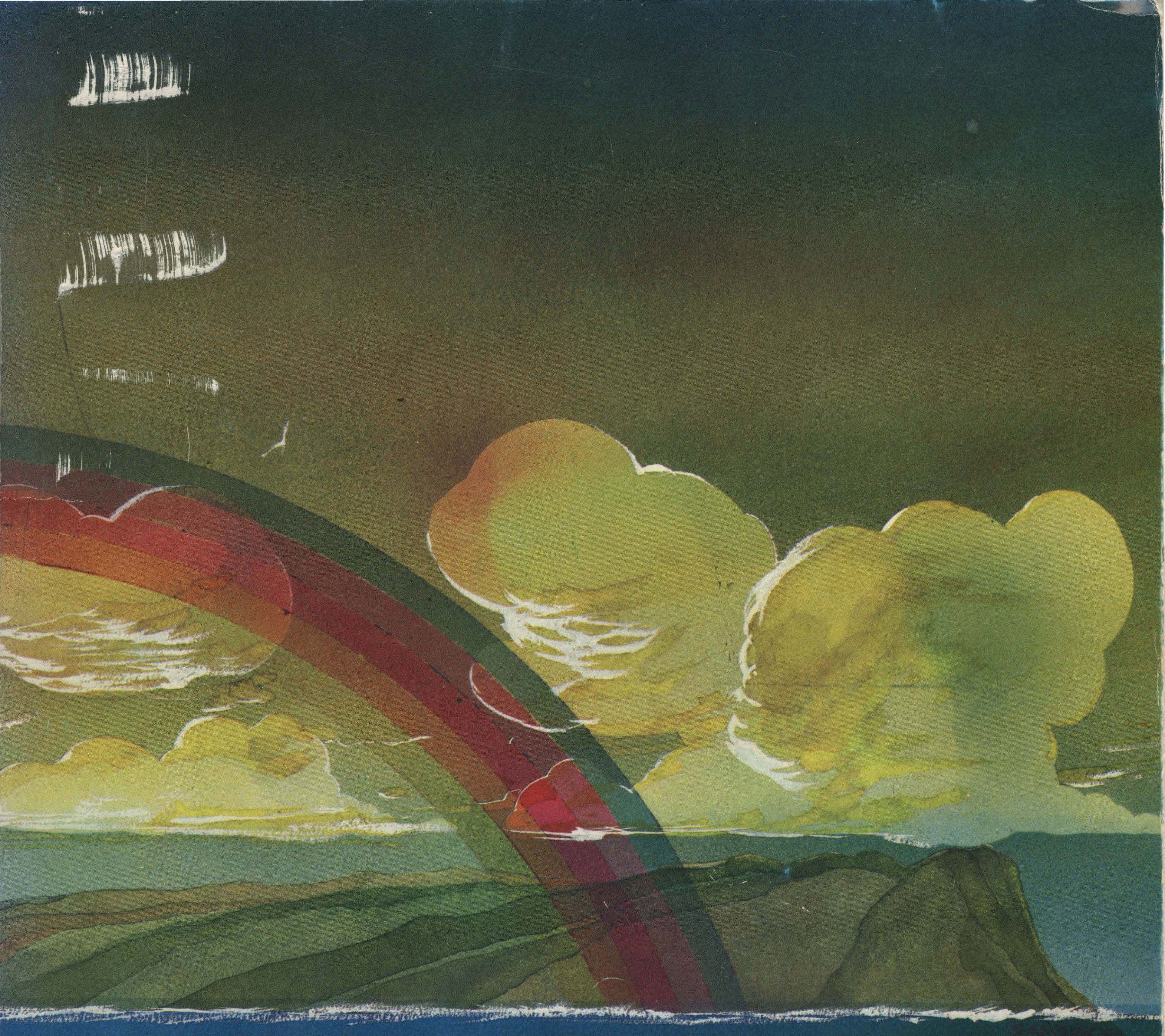

State Of The Groundwater Resources Of Southern Oahu

BOARD OF WATER SUPPLY
CITY AND COUNTY OF HONOLULU

State Of The Groundwater Resources Of Southern Oahu

Board of Water Supply
City and County of Honolulu
1980

[John F. Mink.]

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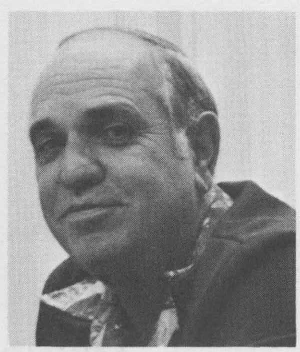

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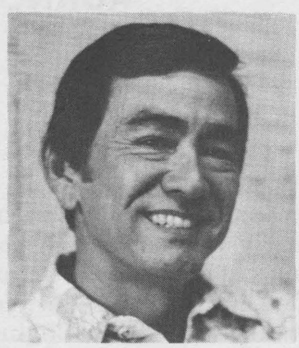

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ACKNOWLEDGMENTS

The project originated when the Manager and Chief Engineer of the Board of Water Supply was Edward Hirata, whose concern about the condition of the water resources of Southern Oahu led to public awareness that these resources were, in fact, limited. The study continued with cooperation and support of the current Manager and Chief Engineer, Kazu Hayashida. Directly overseeing the program has been Herbert Minakami, Chief of Planning and Engineering. The Board of Water Supply staff has unreservedly joined in promoting the objectives of the investigation by providing data and engaging in critical discussions. The members of the Board have shown their deep concern for the water resources of Southern Oahu by affirmatively supporting efforts, including this study, to come to grips with the problem.

This report was written by John F. Mink, who has had a long association with the Board of Water Supply. From 1960 through 1964 he was hydrologist-geologist for the Board, coming from the Honolulu office of the United States Geological Survey where he was a groundwater geologist. From 1957 to 1960 he studied the Pearl Harbor aquifer as a member of the USGS. Mr. Mink has made investigations throughout the Pacific and has made important contributions to the knowledge of the water resources of Guam, the Northern Mariana Islands, Okinawa, Fiji, Tahiti, and numerous other islands, including all of the Hawaiian Islands.

FOREWORD

The Pearl Harbor groundwater basin is much used, much discussed, but less than well understood. It is a major source of water for sugar cane irrigation, diversified agriculture, military, industrial activity and municipal supply. Prior to Statehood, the groundwater resource was ample for all demands placed on it, but with Statehood came rapid economic growth and urbanization. In particular, sugar cane acreage though not decreased in total area, was displaced from the region between Aiea and Waiawa, which became densely urbanized, to lands west of Waiawa, where agriculture now competes with urban sprawl. The effect of these changes has been to markedly increase groundwater draft in the Pearl Harbor basin.

Because the margin between supply and demand for water was ample before Statehood, there appeared to be no pressing need to understand the resource for nearly 60 years. With the large increase in draft of the last two decades, however, there has arisen a need to better understand the nature and limitations of the resource in order to optimally utilize it for beneficial use. The Board of Water Supply has addressed this need ever since it assumed responsibility for all of Oahu in 1959.

The Board of Water Supply was created in response to a water resources crisis in Honolulu that culminated between 1925 and 1930, and since then it has been the principal government agency promoting and engaging in the investigations of the water resources of Oahu. As a supplement to its own staff work, the Board has funded numerous hydrologic and geologic research projects and studies. It maintains an extensive hydrologic data network, the information of which is available to other government agencies and the public. Investigations and data collection, like planning and management, are essential responsibilities of the Board.

Management and planning involve present actions and decisions that affect future developments and activities. To manage and plan intelligently and rationally, it is highly desirable to be able to answer the question: "What would happen if . . .?" Judgment and experience of the managers and planners now play a dominant role in answering the question. A device that can somehow simulate the system being managed would be a valuable tool for managers and planners.

This report in addition to summarizing past experience in developing the groundwater resources of southern Oahu, derives and describes such a device. The mathematical model that has been developed will allow managers and planners to test the effects of various scenarios of demand. For each scenario an ultimate state of the groundwater system is determinable. Good planning and management will decide which scenarios are acceptable.

Planning, management and research are essential to the sustained productivity of the water resources of the Pearl Harbor basin. The model described in this report links these three functions and will be another basic tool to be used by the Board in meeting its responsibilities to the public.

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SUMMARY OF FINDINGS

The study was undertaken to define the state of the groundwater resources of Southern Oahu, in particular those of the Pearl Harbor region, by assessing the results and conclusions of previous investigations, evaluating relevant data, conceptualizing the dynamics of the groundwater system, and constructing an analytical mathematical model that could simulate past aquifer behavior and predict future behavior under different scenarios of development. The findings summarized below include affirmations of conclusions made by other investigators as well as the concepts and understanding that resulted from the study.

1. The aquifers of all of Southern Oahu from Manoa Valley to the Waianae Mountains are hydraulically connected. For convenience the Pearl Harbor region is considered as extending from Red Hill to the Waianae crest and the Honolulu region from Manoa to Red Hill. This arbitrary division has been employed in all hydrologic studies of Southern Oahu.
2. The total natural groundwater flow passing through the Pearl Harbor region lies between 200 and 250 mgd (million gallons per day). This includes infiltration from rainfall and subsurface inflows from the Wahiawa high level aquifer and from the rift zones of the Koolau and Waianae Ranges. It is reasonable to assign a natural flux of 220 mgd to the Pearl Harbor region for modeling purposes. In Honolulu the natural flux is about 60 mgd.
3. The water balance can be used in deriving a working model of the groundwater system if it is cast in terms of natural input (I), draft (D), and leakage (L). Average natural input is considered constant, the draft term appears as net draft (total draft less allowances for return irrigation) in the model equations, and leakage is converted to a head term. The analytical model is based on the functional relationships:

   Transient state: $h = f(I, D, h_o, V_o, t)$
   Steady state: $h = f(I, D, h_o)$

   in which D is net draft, h_o is initial head, V_o is initial storage, and t is time.
4. Although the model is based on simplifying assumptions, in particular a sharp interface between the fresh water lens and underlying salt water, it satisfactorily simulates the historical record and can be used predictively. The model accounts exactly for all component flows in the balance equation at every moment of time.

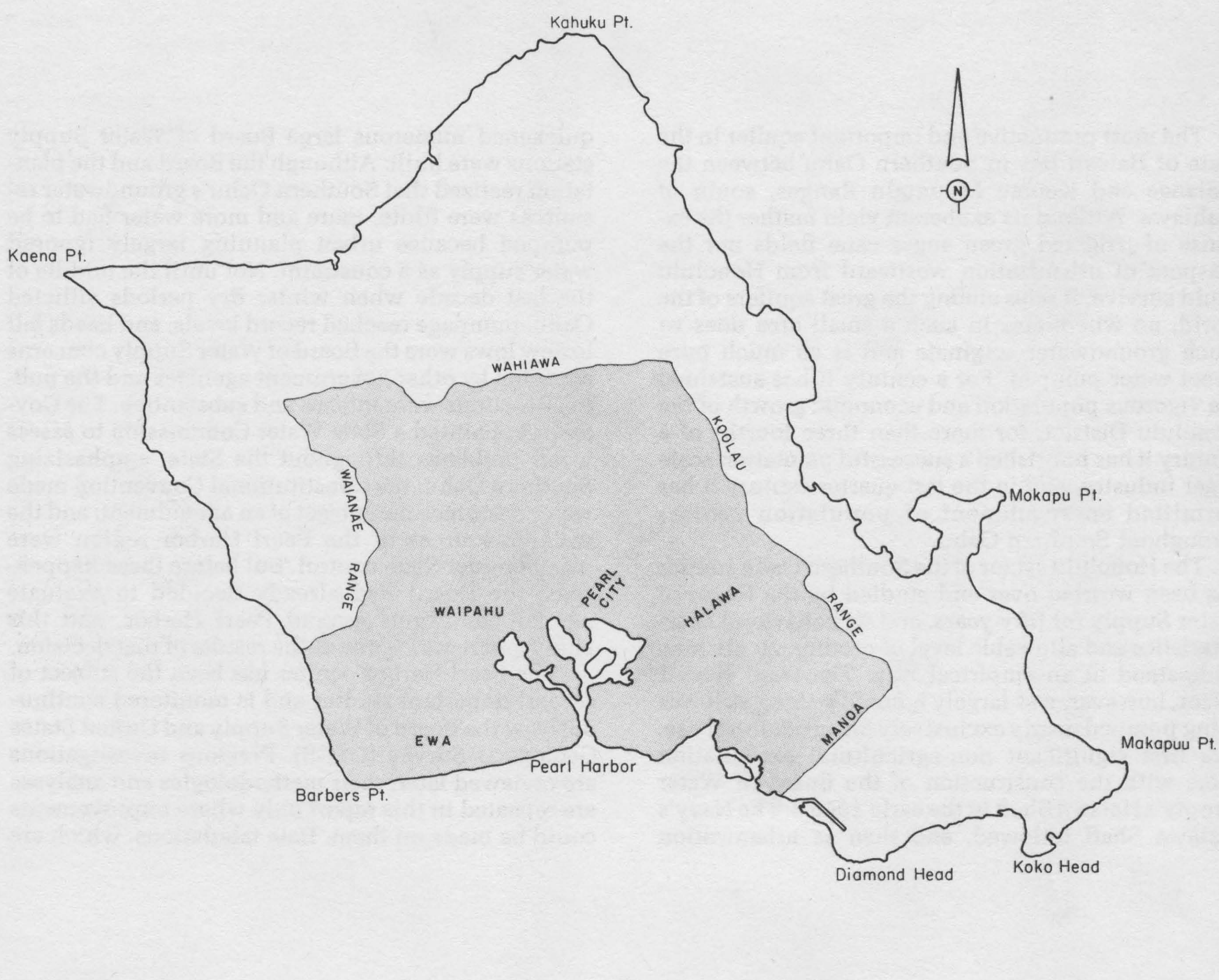
5. The model solves for storage head, which is the index of the volume of water in the basal lens. Storage head is not normally equivalent to the measured water table head, which incorporates drawdown due to pumping. Storage head may exceed operating head by as much as ten feet during maximum pumping periods in the Pearl Harbor region.
6. Results of the model computations support the general appreciation of the nature of the basal aquifers of Southern Oahu and in many instances illuminate difficult concepts, such as Wentworth's doctrine of "bottom storage."
7. Among items made clear by the model are:
   a. The basal aquifers of Southern Oahu are extensive and initially contained a great volume of fresh water, on the order of 3.6×10^{12} gallons. The volume is still large, having been reduced by just 40 percent of its original volume after 100 years of exploitation.
   b. This large storage profoundly affects how the system behaves. Because of its size, the lens contracts very slowly even at high rates of draft.
   c. The behavior of the lens is conservative. As head goes down, leakage decreases quadratically, not linearly. For instance, a twofold reduction in head gives a fourfold reduction in leakage.
   d. Leakage is absolutely controlled by head; draft is arbitrary.
8. The size of the resource allows it to be overdrawn for periods of five to ten years. The extent of the overdraft is governed by a selected transient head to which decline is permitted. Historically net draft plus leakage plus storage loss has exceeded natural inflow.
9. Over the next two decades if net draft is held to its present level and no changes in other hydrologic factors are allowed to take place, in the year 2000 the storage head will be 17 to 18 feet, a loss of about three feet from present storage head. Eventually head would fall to an equilibrium of 14.5 feet.
10. If 45 mgd were added to present net draft in the next 20 years, causing net draft to exceed inflows, storage head would decay to 14 feet by the end of the century but eventually would go to zero. The operating head would be about ten feet less.
11. Sustainable yield depends on equilibrium head. For instance the current net draft of 180 mgd (total draft of 225 mgd less 45 mgd irrigation return) would lead to an equilibrium head of 14.5 feet. If this were the desired equilibrium state of the system, then sustainable yield would be 180 mgd net draft (225 mgd total draft). On the other hand, if the allowable equilibrium head were ten feet, the sustainable yield would be higher.
12. The groundwater system is large and therefore allows for a wide range of management options.
13. Among the management options that could be easily implemented are:
    a. averaging of draft over lengthy periods of about five to ten years.
    b. permitting overdraft for several years at a time, so long as the consequences were comprehended and adjustments were made before transient heads decayed to unacceptable levels.

INTRODUCTION

The most productive and important aquifer in the State of Hawaii lies in Southern Oahu between the Waianae and Koolau Mountain Ranges, south of Wahiawa. Without its exuberant yield neither the expanse of irrigated green sugar cane fields nor the diaspora of urbanization westward from Honolulu could survive. It rates among the great aquifers of the world; no where else in such a small area does so much groundwater originate and is so much pure sweet water pumped. For a century it has sustained the vigorous population and economic growth of the Honolulu District, for more than three fourths of a century it has nourished a successful plantation scale sugar industry, and in the last quarter century it has permitted encroachment of population centers throughout Southern Oahu.

The Honolulu sector of the Southern Oahu aquifer has been worried over and studied by the Board of Water Supply for fifty years, and its behavioral characteristics and allowable level of production are now understood in an empirical way. The Pearl Harbor sector, however, was largely ignored so long as it was being pumped nearly exclusively for agricultural use. The first significant non-agricultural exploitation came with the construction of the Board of Water Supply's Halawa Shaft in the early 1940's. The Navy's Waiawa Shaft followed, and then as urbanization quickened numerous large Board of Water Supply stations were built. Although the Board and the plantation realized that Southern Oahu's groundwater resources were finite, more and more water had to be pumped because urban planning largely ignored water supply as a constraint. Not until the middle of the last decade when winter dry periods afflicted Oahu, pumpage reached record levels, and heads fell to new lows were the Board of Water Supply concerns accepted by other government agencies and the public. Reactions were intense and substantive. The Governor appointed a State Water Commission to assess water problems throughout the State, emphasizing Southern Oahu; the Constitutional Convention made water resources the subject of an amendment; and the water resources of the Pearl Harbor region were placed under State control. But before these happenings, the Board had already decided to evaluate aquifer conditions around Pearl Harbor, and this study and report is one of the results of that decision.

The Pearl Harbor aquifer has been the subject of several important studies and is monitored continuously by the Board of Water Supply and United States Geological Survey (USGS). Previous investigations are reviewed later; their methodologies and analyses are repeated in this report only where improvements could be made on them. Data tabulations, which are



Southern Oahu Study Area

essential to hydrologic and hydraulic models, are included only to the extent that they clarify conclusions. The data for Southern Oahu is efficiently cataloged and graphed in Board of Water Supply and USGS reports.

The study was structured to subsume applicable results of earlier investigations, but it emphasizes development of a model of the dynamics of the groundwater system that simulates the historical record and could be used to predict future aquifer behavior under different scenarios of development. A principal product is a straightforward quantitative analytical model that correctly describes aquifer behavior and greatly reduces the element of speculation about the actual state of the system. The model is robust but consistent, and it is especially suitable as a tool for management.

By and large the objectives suggested in the original proposal for the study have been achieved. The objectives and the extent of their achievement are as follows:

1. Define the water resources system: this objective has been accomplished.
2. Identify flow paths and quality parameters: flow paths are included in exposition of the dynamics of the system, but quality parameters have not been addressed other than superficially because the focus of the study has been on describing hydraulic behavior of a large basal lens under development stresses. Quality considerations are not yet as amenable to mathematical quantification as are hydraulics. Degradation in quality will become a primary concern as the lens thins.
3. Construct hydrological balances that include both flow and quality components: the flow components have been balanced by several different methods; quality was not addressed.
4. Create a dynamic model of the system: a robust analytical model has been derived.
5. Determine steady states (and transient states where appropriate) for various scenarios of development: accomplished.
6. Select realizable equilibrium states in which optimal development could take place: this objective is satisfied by the determination of sustainable yields.
7. Make an inventory of groundwater stored in the aquifers: accomplished.

The report is written to be read and comprehended by planners, managers, engineers and interested laymen. Tedious analyses and data assessments that support statements made in the main text are included in appendices. Not all mathematics could be avoided in the narrative, but what is there could be bypassed without damage to understanding the points made.

Interstate Highway in the Pearl Harbor area superimposed over existing road system, after urbanization.

REVIEW OF PREVIOUS WORK

In recent years a great deal of attention, not all of it supported by investigation, has been given to the water resources of Southern Oahu, in particular the Pearl Harbor region. Before 1955, however, surprisingly little concern was shown for this greatest of all Hawaiian aquifers, except for its Honolulu portion. In Honolulu the Board of Water Supply and its predecessors were continually engaged in evaluating the sub-area aquifers and reported their conclusions in many reports and memoranda. West of Halawa Stream the plantations had successfully exploited the resource since the turn of the century without either reducing its reliability or diminishing its utility and the system seemed to be at equilibrium. Not until the urban invasion of the shores of Pearl Harbor at about the time of the granting of Statehood did the Board of Water Supply, the U.S. Navy and the plantations begin to question whether the resource was large enough to satisfy all the demands implicitly being projected on it.

The first regional investigation in response to this concern was undertaken by the USGS in cooperation with the City and County of Honolulu. This study (F. N. Visher and J. F. Mink, 1964) was a transition between the earlier regional work of Stearns and Vaksvik (1935, 1938), Stearns and Macdonald (1940), and Wentworth (1942, 1945, 1951). Follow up regional studies were made by the USGS (Dale, 1967; Dale and Ewart, 1971; Soroos and Ewart, 1979) and by the BWS (unpublished memoranda, BWS files).

Stearns and Vaksvik (1935) laid out the geologic framework of Southern Oahu, made preliminary groundwater evaluations and, very importantly for later investigations, compiled hydrologic data for the period of record extending back to the turn of the century, including rainfall and draft. Stearns had made a brief study of the Pearl Harbor springs in 1931, but before that few instructive references to the hydrology of the region appear in the literature.

Schuyler and Allardt (1889) discussed the springs in a report concerned with water for irrigation of Honouliuli lands, and occasional comments about the number of wells drilled and water pumped are scattered in memoranda and company reports. It is a lasting credit to the plantations that they kept records of draft and frequently measured and recorded heads, and to the Territory Division of Hydrography and the USGS that they established benchmark data points such as observation wells. Once the BWS extended its jurisdiction to the Pearl Harbor region in 1959, it set up a comprehensive data collection network.

In a first detailed look at a portion of the Pearl Harbor region, Stearns in 1931 concluded that the springs were effectively artesian, a correct deduction but one questioned by Wentworth who at first believed they were caused by horizontal flow along the top of the water table. Stearns reported a measured flow of 66 mgd, not greatly different from the estimate of 75 mgd made by Schuyler and Allardt in 1889 or the average of 87 mgd made by Visher and Mink for 1953-1957. Both the Schuyler and Allardt, and the Stearns figures are low for their time, as perhaps so was that of Visher and Mink. The 1935 report of Stearns and Vaksvik (Bulletin 1 of the Division of Hydrography) was the landmark in establishing a rational hydrogeologic framework for all of Oahu. Aside from discussing the geology of Southern Oahu, its most important contribution to comprehending the Pearl Harbor groundwater resources lay in its compilations of head, draft and chlorides and the discussions of their significance.

The BWS became interested in the eastern Pearl Harbor region as a source of water for Honolulu about the time of the second world war. Wentworth reported on his investigations of the Moanalua-Halawa district in 1942 and completed his initial studies of the Pearl Harbor district in 1945. He calculated a detailed hydrologic budget for the district, unsuccessfully attempted to establish a definitive correlation equation among draft, rainfall and head, and commented on regional aspects of groundwater hydraulics. He also described the geology more fully than had Stearns. In 1951 he enlarged upon his original observations. In evaluating the water resources he concentrated on multiple correlation (head, draft, rainfall), hydrologic budgeting and the concept of bottom storage. A critique of his evaluations is given elsewhere in this report.

Wentworth was very concerned that more water was being extracted from the Pearl Harbor aquifer than was being recharged and that the difference was provided by bottom storage, a finite volume that he considered near exhaustion in 1940. He stated (1951, p. 99) "the draft of as much as 250 mgd from an area not known to receive more than about 369 mgd (i.e., rainfall) represents a remarkably large percentage and is probably to be explained in part by yield from shrinking storage." He made an interesting comment (p. 101) that a seasonal minimum head might be as low as nine feet at 50 to 100 years in the future, a rather optimistic opinion from our contemporary point of view but surely meant as a foreboding one. Wentworth's work was filled with imaginative approaches that should interest every hydrologist. He was not the only one at the Board of Water Supply trying to comprehend the vast aquifers of Southern Oahu, however. As a result of a noteworthy collection and abstracting effort on the part of L. J. Watson, excellent files that include relevant memoranda and other forms of communication dealing with Southern Oahu are maintained at the Board of Water Supply.

The Visher and Mink report (1964) scrutinized the hydrologic changes that had taken place between 1910 and 1960. The objective was to determine a safe (sustainable) yield for the Pearl Harbor region. They attempted to establish behavioral features of the resource by studying the history of draft, heads and chlorides and to establish aquifer parameters from drawdown and recovery tests. Qualitative hydraulics of a basal lens, including the mechanism of spring flow, were derived, and the geochemical balances in the hydrologic cycle, especially with respect to the effect of return irrigation water, were rationalized. They concluded that about half the irrigation applied over the basaltic aquifers percolates to the saturated zone, that sea water intrusion was not overtly active, and that the prevailing average draft of 160 mgd was less than sustainable yield. They estimated that about 50 mgd additional draft could be safely withdrawn. The investigation was completed just as intensive urbanization began in the district. The 50 mgd additional draft considered allowable was to be appropriated within the following decade.

In 1967 Dale of the USGS updated the Visher and Mink study, focusing attention on changes in land use, and therefore water allocations, that had taken place since 1931. Through the use of outflow computations he concluded that the total groundwater discharge from the Pearl Harbor aquifer averaged 250 mgd between 1931 and 1965. A further update was made by Dale and Ewart in 1971 and most recently by Soroos and Ewart, (1979). In the latter report it was inferred that over the period 1910-77 an average of 25 mgd of groundwater from the original storage of the lens contributed to the total discharge.

Other investigations of areas within the Pearl Harbor region have been made by staffs of the BWS and the Water Resources Research Center of the University of Hawaii (Lau, 1961; Hufen, 1973). Quality and volumes of water were discussed in the Oahu Water Quality Program (1972). A surface water assessment was made by Hirashima (1971) and recently by R. M. Towill for the U.S. Army Corps of Engineers (1978).

Results of all of the previous investigations were considered during the present study and many are woven into the analyses, evaluations and conclusions that make up this report. Significant contributions are usually credited. However, in this study I have attempted to shift the focus from description and compilation to analysis of the non-equilibrium state of the groundwater system, and therefore many of the techniques are new for the Pearl Harbor region and at least one is unique.

Reports

HYDROLOGIC BUDGETS

Hydrologic budgeting is the favorite method for gaining an understanding of the limitations of a water resource. But it is a static approach that fails to take into account the dynamics of the hydrologic cycle, describing instead an averaged equilibrium state that does not contend with the transitional phenomena which in a basal groundwater resource as large as that of Southern Oahu occur over long time intervals. C. K. Wentworth wrestled with this problem by proposing his concept of "bottom storage," in which delayed yield from deep in the basal lens moves toward extraction sites for many years following original head decay. In a sense the "bottom storage" phenomenon is real, but neither in the hydraulic fashion nor as a latent provider of pumped water as envisioned by Wentworth.

Hydrologic budgets, in spite of their limitations, are valuable and often essential in setting initial and boundary conditions of a groundwater resource, and because of this important application they have been the focus of numerous groundwater investigation in Southern Oahu. Budgets are composed by computing water balances under natural or development conditions, by combining the balance method with Darcy's Law, and by analyzing apparent equilibrium states during sustained constant pumping. The balance method matches input (rainfall, underflow and surface water diversion) against output (evapotranspiration, runoff to the sea, draft and leakage). Darcy's Law gives specific flow as the product of hydraulic conductivity and gradient. The equilibrium state method assumes that if a constant head is maintained at constant pumping the draft is equivalent to recharge; it is the least accurate of the three methods because it ignores transient leakage.

The simplest and most common hydrologic budget deals with only natural inputs and outputs of the water system being described. The method assumes that topography defines the groundwater boundaries so that a balance exists among rainfall, direct surface runoff out of the basin, evapotranspiration and infiltration. The balance equation is simple,

(1) $I(P) = P - R - E$

in which I(P) is infiltration from basin rainfall, P is rainfall, R is direct surface runoff and E is evapotranspiration. To this simple balance must be added subsurface underflows across the topographic boundaries to obtain total natural recharge,

(2) $I = I(P) + I(U)$

wherein I is total natural groundwater recharge and I(U) is subsurface inflows. In this elementary balance, total recharge is equal to total leakage, L. In the above equations change in storage is not considered because steady state is assumed.

A second approach is to write the balance between inputs and outputs to the groundwater system under conditions of its development. For the Pearl Harbor region the general equation is:

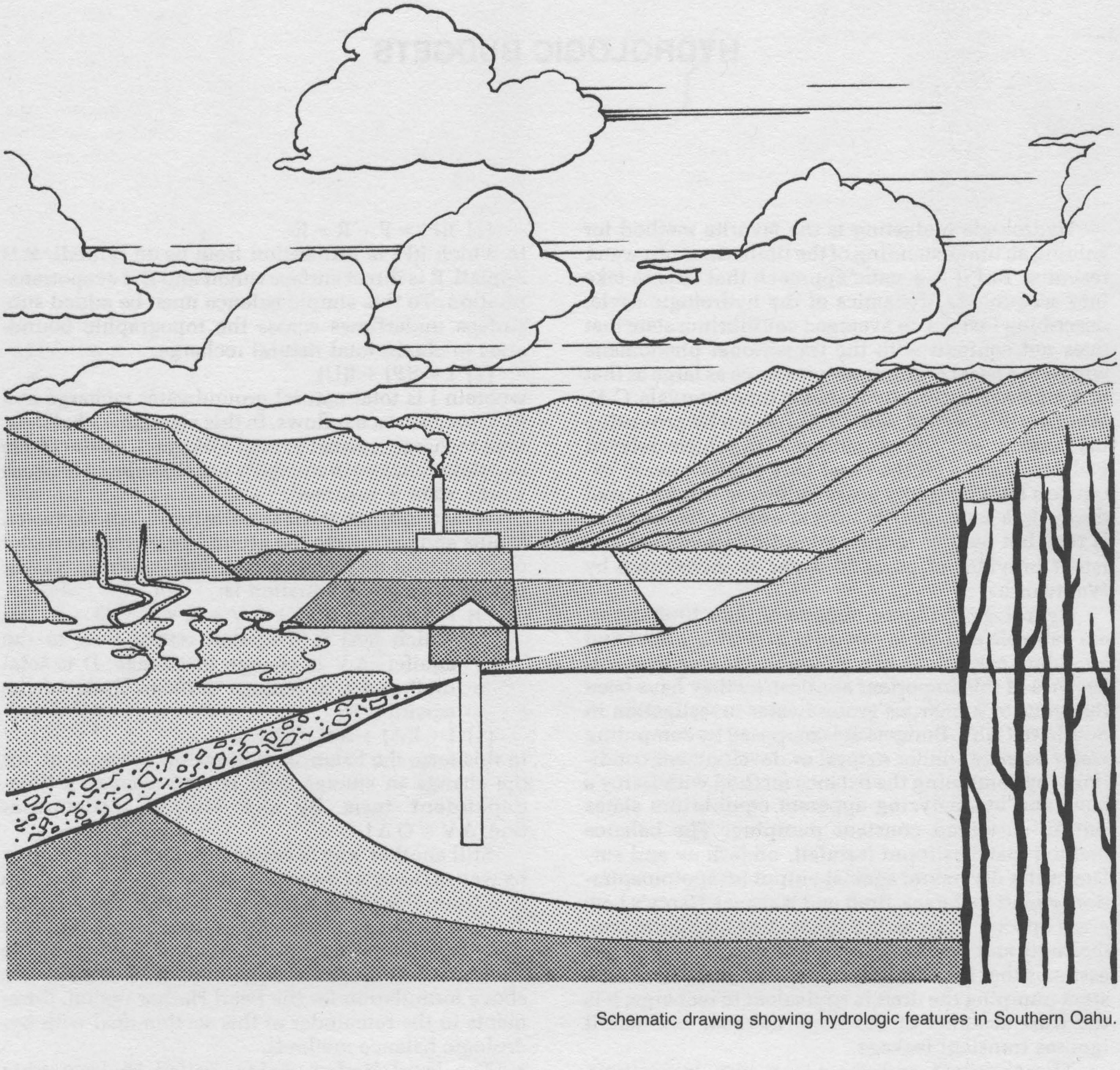
(3) $P + I(U) + I(A) + \Delta V = R + E + D + L$ in which I(A) is irrigation return flow to the aquifer, ΔV is change in storage, D is total draft and L is leakage outflow. Restated the equation is:

(4) $I + I(A) + \Delta V = D + L$

In this form the balance relationship is transient, for the change in storage term is converted to a time-dependent term by the simple transformation, $\Delta V = Q \Delta t$.

Still another way to estimate groundwater flow is by way of Darcy's Law expressed as Q = TiL, in which Q is flow, T is transmissivity, i is hydraulic gradient and L is width of section. This form assumes steady state conditions during the interval for which the computation is made. Appendix II discusses the above formulation for the Pearl Harbor region. Comments in the remainder of this section deal with hydrologic balance methods.

The Pearl Harbor region, called Hydrographic Area IV in the State Water Resources Development Plan (1980) and earlier State publications, extends from Red Hill and the Koolau crest to the Waianae crest, and from the topographic limit of the Wahiawa plateau to Pearl Harbor and the Pacific Ocean. Not all compilers of water balances have standardized on



Schematic drawing showing hydrologic features in Southern Oahu.

these boundaries, however. A common alternative is to choose the Koolau basalt-Waianae basalt unconformity, extending from Ewa to the Wahiawa high level aquifer, as the western boundary, eliminating about 25 sq. miles and 40 mgd of rainfall. More recently Broadbent (1980) chose boundaries along Waimalu Valley on the east and included a large section of the northward draining Kaukonahua on the northwest. This lack of uniformity of boundaries confounds comparisons among computed budgets.

Voorhees, as reported in Stearns and Vaksvik (1935), made the first of numerous rainfall volume computations for the region. Based on records until 1928, which gave slightly higher rainfalls than has the longer term average, he calculated an average annual rainfall volume of 492 mgd on the 168.92 sq. miles of Hydrographic Area IV. This total includes the caprock and Waianae basalt areas. For the Koolau portion of the region Wentworth (1951) computed rainfall of 420 mgd, but only 369 mgd for the "intake area," the sector overlying the basalt aquifer. In the Hawaii Water Authority publication (1959), Water Resources of Hawaii, a total median annual rainfall volume of 509 mgd for 178 sq. miles was assigned to Hydrographic Area IV. Both the area and the rainfall are greater than measured by other investigators. The median rainfall was used but it differs little from the average because annual rainfall in Hawaii closely follows the normal distribution. In a budget prepared for the USGS Pearl Harbor study (Visher and Mink, 1964) but not published, Mink computed 428 mgd for the total area over basaltic aquifers (Waianae plus Koolau) by employing the exponential equation of rainfall increase for regions above the 60-inch annual isohyet and arithmetic averaging for lower rainfalls.

If caprock rainfall is subtracted from Voorhees' total of 492 mgd the intake balance is 448 mgd, and if the Wentworth computation is standardized by adding the Waianae basalt sector his intake value would be 409 mgd. Dale (1967) gave a figure of 400 mgd but did not specify whether the caprock was included. The State Water Resources Development Plan, based on the Hawaii Water Resources Regional Study and other studies, gives a total of 425 mgd, though it is not clear if the caprock is included. However rainfall has been computed, it is reasonable to conclude that over the basalt, Waianae plus Koolau, within Hydrographic Area IV the annual average is more than 400 mgd and less than 450 mgd, and therefore a value of 425 mgd is the best estimate.

Measurements are made of stream flow in the Pearl Harbor region but only for about 65 percent of the non-caprock area. Before stream gages were installed, Wentworth (1951) estimated direct stream runoff of 98 mgd, which is too high. Hirashima (1971) summarized flow data for the 83.3 sq. miles monitored with continuous recording gages. He reported average direct runoff from this area as 42.5 mgd; a proportional correction for the entire non-caprock portion of the basin yields a total average of 65.4 mgd. The State Water Resources Development Plan assigns a direct runoff of 70 mgd to the harbor. For the U.S. Army Corps of Engineers' study of impoundment possibilities in the Pearl Harbor region (R. M. Towill, Inc., 1978) Mink calculated a total direct runoff from basalt (Waianae plus Koolau) of 61 mgd by updating Hirashima's work and making supplementary analyses. This amounts to 14 percent of total rainfall. The average value of direct runoff evidently lies between 60 and 70 mgd.

The other natural output component, evapotranspiration, is the most difficult to estimate because its behavior in the wet mountains is conjectural. Wentworth (1951) estimated a total evapotranspiration of 100 mgd for the Koolau basalt intake area, equal to 27 percent of rainfall. The State Water Resources Development Plan estimates 105 mgd for Hydrographic Area IV, or 25 percent of rainfall. Mink (unpublished budget) computed evapotranspiration separately for areas receiving more than 60 inches rain per year and for areas of less than 60 inches. For the wet mountains he assumed evapotranspiration to be inversely proportional to rainfall, equivalent to exponential decay of evapotranspiration with distance from the 60-inch isohyet to the point of maximum rainfall just leeward of the Koolau crest. Above the 60-inch isohyet evapotranspiration was computed as 79 mgd, equal to 28 percent of rainfall; for the lower rainfall sector evapotranspiration was computed as 75 mgd, or 50 percent of rainfall. Total evapotranspiration for the basalt portion of the basin added up to 154 mgd, 36 percent of rainfall, which is appreciably higher than either Wentworth's estimate or that of the State Water Resources Development Plan.

Natural infiltration to groundwater is taken as the difference between average rainfall and average direct runoff plus evapotranspiration. For each evaluation reviewed above, the components of the balance equation are as follows on Table 1.

TABLE 1

Natural Hydrologic Budget
Pearl Harbor Region Total Basalt Area (mgd)

Source	Average Rainfall	Average Direct Runoff	Average Evapo-transpira-tion	Average Infiltra-tion	Remarks
Wentworth, 1951	409	98	100	211	Original values corrected to include Waianae basalt
Mink	428	61	154	213	Total basalt area
State Water Resources Development Plan	425	70	105	250	Values assumed to refer only to basalt area

Actual average natural infiltration from rainfall is probably greater than 200 mgd yet less than 250 mgd. Most likely it falls between 200 and 225 mgd, but this is judgment, not the conclusion of precise analysis. The infiltration does not include subsurface inflow. However, for the Koolau rift zone, the Wahiawa high level water and the Waianae rift zone, the subsurface flow is estimated to be about the same as the rainfall infiltration on the areas overlying the portions of these water bodies falling within the topographic drainage of the Pearl Harbor region. Subsurface flow from the Honolulu District is separate and must be added to rainfall infiltration to give total natural groundwater input to the region.

The development budget is more complicated than the natural budget, yet if all outflow can be measured or estimated, including the loss from storage, the total groundwater flux of the system is determinable without having to wrangle with the uncertainties of direct runoff and evapotranspiration. In equation (4), draft, D, is known, leakage, L, can be approximated from spring flow and seepages, return irrigation, I(A) can be estimated, and long term average change in storage, ΔV, can be established from the slope of declining head, leaving natural total recharge, I, as the single unknown. All of the estimates are afflicted with imprecision, however. Later, as part of the derivation of a mathematical model of the groundwater system, it will be shown that the leakage term can be converted to a precise head and that the storage term may also be parameterized in terms of head.

A good estimate of the return irrigation component is possible because flows to the fields are known, evapotranspiration during various stages of growth has been accurately measured, and direct runoff is negligible. A generally applied rule equates pan evaporation with evapotranspiration, though this equivalence is strictly applicable only to the early vigorous period of growth. Later in the growth cycle evapotranspiration is less than pan evaporation by about 25 percent.

Return irrigation is meaningful only where it takes place above the basaltic aquifer. Furrow irrigation, the dominant technique until a few years ago, yields more infiltration than does the drip method. Averaged to a daily basis, 8,000 to 10,000 gallons per acre per day is required by the furrow method. Dale (1967) calculated an average of 9,000. Whereas about 50 percent of water applied to furrows percolates beyond the root zone, for drip irrigation the proportion is nearer 25 percent. A return irrigation balance in the Pearl Harbor region overlying the aquifer, prepared for the Oahu Water Quality Program (1972) by Mink, is given below. The balance assumes furrow irrigation, the prevailing practice when it was made.

TABLE 2

Pearl Harbor Region Return Irrigation Flows
Averages Recalculated to Gallons Per Acre Per Day (gpa/d)

Component	Input in/yr.	Input gpa/d	Output in/yr.	Output gpa/d	Remarks
Rainfall	35	2,600			
Non-effective Rainfall			14	1,040	40 percent total rainfall
Direct Runoff			3	220	9 percent total rainfall
Effective Rainfall	18	1,340			51 percent total rainfall
Irrigation	121	9,000			
Total avail. Water	139	10,340			
Evapotranspiration			70	5,200	Equal to pan-evaporation
Recharge	69	5,140			49.7 percent of avail. water

Notes:

1. Recharge: 49.7 percent total available water
2. Recharge from rainfall: .497 × 1,340 = 667 gpa/d
3. Recharge from irrigation: .497 × 9,000 = 4,473 gpa/d
4. Total irrigated acreage over aquifer: 10,000 acres
5. Total infiltration to aquifer from irrigated acreage:
   a. Grand total: 5,140 × 10,000 = 51.4 mgd.
   b. Rainfall component: 667 × 10,000 = 6.7 mgd
   c. Irrigation component: 4,473 × 10,000 = 44.7 mgd
      1. Waiahole Ditch: .497 × 30 = 14.9 mgd
      2. Draft: 44.7 − 14.9 = 29.8 mgd

Dale (1967), Soroos and Ewart (1979), and Broadbent (1980) employed gross outflow values for estimating groundwater flux and components of that flux. Dale computed balances for two periods, as follows (values in mgd):

Period	Draft plus Spring flow	Return Irrigation	Net Input
1931-32	250	40	210
1964-65	250	30	220

The above assumes equilibrium conditions and therefore no loss of storage, an unlikely event. Allowing a storage loss would decrease the calculated net input. The balances of Soroos and Ewart included average storage depletion consistent with the long term decay in aquifer head. Their results for the period 1910-1977, modified by assuming return irrigation of 45 mgd, are:

Draft plus spring flow	275	mgd
Loss in storage	25	mgd
	250	mgd
Return irrigation	45	mgd
Net input	205	mgd

In both of the above balances it is implicitly assumed that total leakage is identical with measured spring flow; if this were not the case and more leakage occurs, such as into the caprock, the calculated net input is too low.

Broadbent, whose computation area differs appreciably from the above investigators, also included a storage loss component. He computed an unused leakage of 77 mgd from the Pearl Harbor region by subtracting outputs (exports from the basin 113 mgd, direct runoff 67 mgd, total evapotranspiration 212 mgd) from inputs (rainfall 433 mgd, storage loss 19 mgd, net Waiahole Ditch return irrigation 17 mgd). His leakage value is similar to the total spring flow used by the USGS investigators.

No matter how the development balances are structured, the natural input is computed as about 200 to 225 mgd, consistent with results of the natural budget method. The complexities of the groundwater system in Southern Oahu are shown in Figure 1 as a flow diagram, which is based upon measured values where possible and inferred values otherwise.

In the robust analytical model a value for natural input to the Pearl Harbor region is needed. The value selected for the combined Koolau and Waianae basalt sectors from Red Hill to the Waianae crest is 220 mgd, which is consistent with the natural input derived from both the natural and development hydrologic balances.

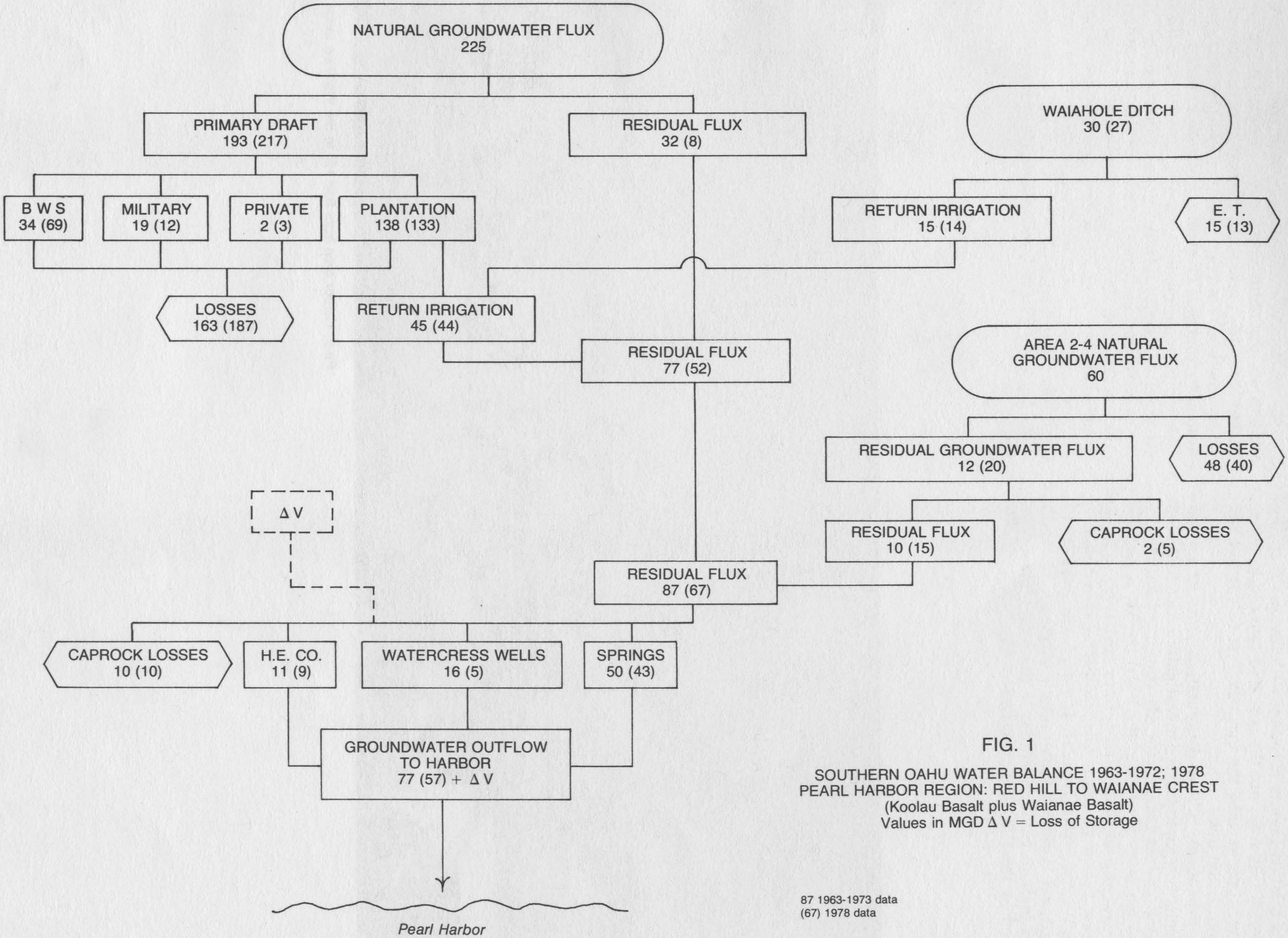
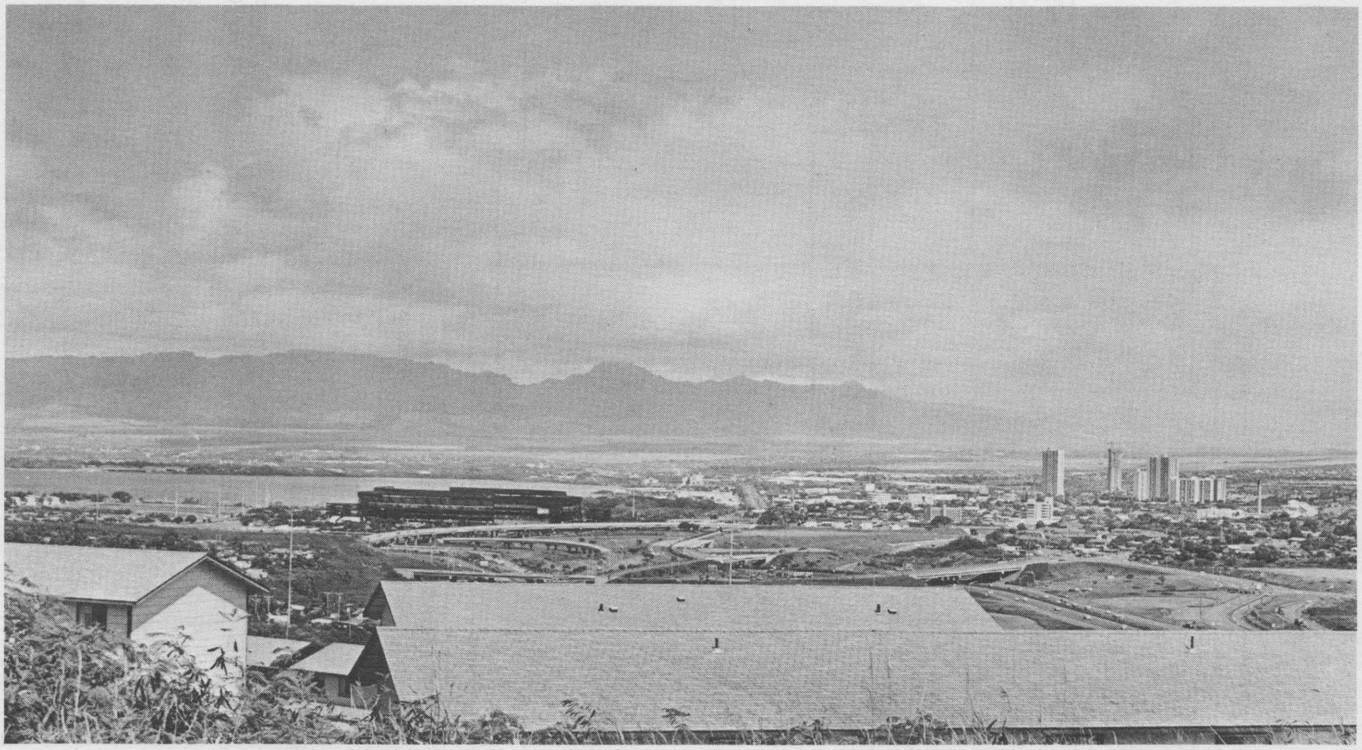


FIG. 1

SOUTHERN OAHU WATER BALANCE 1963-1972; 1978
PEARL HARBOR REGION: RED HILL TO WAIANAE CREST
(Koolau Basalt plus Waianae Basalt)
Values in MGD Δ V = Loss of Storage

87 1963-1973 data
(67) 1978 data



Pearl Harbor area from Red Hill to the Waianae range.

RELATIONSHIP BETWEEN THE HONOLULU AQUIFER AND THE PEARL HARBOR AQUIFER

The basal groundwater resources of the Honolulu District traditionally have been treated as separate from the Pearl Harbor aquifer even though flow from the Moanalua region, Area 4 of Board of Water Supply terminology, freely moves into the Halawa region, the start of Area 6. In fact, all of Honolulu west of Manoa Valley is hydraulically connected to the Pearl Harbor basal lens. The entire region extending from Manoa to the Waianae crest and from the caprock wedge to the Koolau and Wahiawa high level zones is a single, though geometrically complicated, basal aquifer. Only for convenience has a division been made at Red Hill.

The highest heads are in Honolulu Area 2, which is bounded on its east by the deep alluvial fill of Manoa Valley and the Koolau rift zone that bulges into the valley. Some flow from Area 2 may move to Area 1 (Kaimuki) just east of it, but the prevailing direction of groundwater movement is westward to Area 3 (Kalihi), which is continuous with Area 4. The Honolulu subregions have been called "isopiestic areas" because heads apparently are identical within each subregion (the word "isopotential" would have been more appropriate than isopiestic). If indeed the heads were identical over a region, groundwater would not flow, an impossible situation given that inflow to the aquifers must be balanced by outflow. Actually, throughout the Honolulu District groundwater flows northwestward along a gradient of one foot to the mile. The equipotentials generally are aligned at right angles to the trace of the caprock, curving to be continuous with those in the Pearl Harbor region west of Moanalua (see equipotential maps).

The groundwater of the Honolulu District that is not extracted by pumps or does not leak into the caprock flows toward the Pearl Harbor springs. At the high original heads, leakage in Honolulu was principally at the inner thin edge of the caprock; as the heads were reduced by pumping, this leakage diminished, though even today it persists. Probably very little leakage penetrates the clays and compacted terrestrial alluvium at the base of the thicker part of the caprock wedge. The residual flow from Area 4 toward the springs is on the order of 10 to 15 mgd under current development practices. Except perhaps for the short interval when Shaft 12 (Halawa) and Shaft 11 (Red Hill) were being dewatered during excavation, the groundwater flow gradient has always been from the Honolulu District toward Pearl Harbor.

The rift zone of the Koolau Range forms a precise boundary to the Pearl Harbor-Honolulu aquifer. The boundary generally lies about one half mile leeward of the crest of the range. Groundwater from high level dike aquifers leaks to the basal aquifer of leeward Oahu as well as to tunnels and springs on the windward side of the crest. In hydrologic budgeting, rainfall in the rift zone on the lee side of the crest is assumed tributary to Southern Oahu and its infiltration component is considered equivalent to subsurface flow from the high level aquifers.


Pearl Harbor area looking southeast from Wahiawa.

RELATIONSHIP BETWEEN THE WAHIAWA HIGH LEVEL AQUIFER AND THE PEARL HARBOR AQUIFER

The northern extension of the Pearl Harbor basal lens apparently is abruptly terminated just to the south of Wahiawa by a structure that impounds a high level aquifer having a water table 250 feet higher than the basal lens. The structure is probably a buried rift zone striking eastward from the Waianae caldera, but to date no physical evidence of it has been identified. The boundary must be relatively sharp because Well 250-2 in the basal aquifer lies only a mile south of Shaft 4 in the high level aquifer. To the west, where Kunia Road crosses the Waianae drainage of Waikele Stream, the high water table at Well 330-7A descends to basal water at Well 330-5 lying less than a mile to the south. Several wells and test holes just north of 330-7A show that the high level water surface decreases in steps rather than along a continuous gradient.

A reliable estimate of the rate of subsurface flow of the high level water to the Pearl Harbor aquifer is impossible to make with the very limited data set on hand and the absence of a rational model to simulate the conjunctive behavior of the aquifers. Dale and Takasaki (1976) attempted to compute a hydrologic budget for the higher aquifer, to model its hydraulics and to estimate groundwater discharges from it to the north and south. They considered the high level area as 34 sq. miles and infiltration into it of 127 mgd. Of this total they assigned a flow of about 100 mgd to the Pearl Harbor aquifer. Hydrologic budgets and inflow-outflow computations of the sort from which the above figures were derived are so imprecise as to be meaningful only in the most approximate sense. The attachment of specific numbers to rates does not refine the highly qualitative nature of the methods. Until the physical boundaries and better comprehension of the subsurface environment and the dynamics of the water resources have been ascertained, it is at least as rational to include subsurface flow to the south only the infiltration fraction of the rain that falls on the high level sector within the topographic boundaries of Southern Oahu. This artifice was employed in all hydrologic budgets made during the present study.

Pearl Harbor area looking east from the Waianae Range.

WAIANAE BASALT SECTOR OF THE PEARL HARBOR AQUIFER

Stearns and Vaksvik (1935) noted that heads at Oahu Sugar pumping station 5 (Well 274) were lower by a significant amount than heads further east in Honouliuli and that changes in head in the two sectors did not follow the same pattern. The differences in behavior were attributed to a partial groundwater barrier between the Koolau and Waianae lavas. Stearns concluded that the barrier is the erosional unconformity, consisting of a weathered zone and accumulations of alluvium, separating the lower, older Waianae volcanic series from the younger Koolau volcanic series. There is no doubt that this is indeed the case, but nevertheless hydraulic continuity exists between the aquifers and for this reason they are regarded in combination as the Pearl Harbor aquifer.

The demarcation between the Waianae and Koolau aquifers has been approximated as lying along the exposed surface contact of the two formations. The effective separation lies further eastward because the Waianae series dips from five to ten degrees beneath the Koolau rocks in this direction. The sea level contact is about a mile to the east of the surface evidence. On the west the Waianae aquifer is abruptly terminated along a line passing up Makaiwa Gulch to the southern Waianae rift zone. On the north it is terminated by the boundary of the Wahiawa high level water. Total area of the aquifer, including the caprock portion, is about 35 sq. miles; the non-caprock sector has an area of about 25 sq. miles.

Head drop immediately across the unconformity is two to two and a half feet. This estimate was made by comparing simultaneous maximum heads of wells in Honouliuli with Wells 274 and 276 and Test Holes T-19, T-20 and T-4 in the Waianae aquifer. Correcting for the normal hydraulic gradients of 1.0 feet per mile in the Koolau aquifer and 1.3 feet per mile in the Waianae gives the head loss caused by the unconformity. The value of five feet estimated by Stearns includes head reduction resulting from the normal hydraulic gradient in each aquifer.

The unconformity is a slight impediment but not a barrier to groundwater flow. The Makaiwa boundary, on the other hand, apparently is a nearly absolute barrier. Two test bores, 0.8 miles apart and straddling the boundary, consistently showed head difference of nine to ten feet during their period of simultaneous head record (1953-1972). The average head of T-4 on the Ewa side of the boundary was 13.7 feet and that of T-5 on the Kahe side was 4.4 feet.

Between the two boundaries, a leaky one on the east and an essentially closed one on the west, the Waianae aquifer lies as a two to four mile wide strip striking northward for about nine miles between the poorly permeable caprock and the essentially impermeable high level boundary. Flow moves southwesterly from the unconformity, and the residual that is not extracted by plantation pumps (Wells 274 and 276) and other pumping stations (Shaft 14 and the Makakilo Quarry Well) leaks into the caprock. A much smaller quantity may move across the Makaiwa barrier toward Kahe. Leakage has always been through the unconformity from the Koolau aquifer into the Waianae aquifer; there is no evidence that flow has ever been reversed. If the Koolau were considered a unit aquifer, its leakage would be apportioned among the Pearl Harbor springs, flow through the unconformity, and seepage into the inner edge of caprock in the Honouliuli area.

The natural recharge over the basalt portion of the Waianae aquifer, computed by hydrologic budget methods explained elsewhere, averages about 20 mgd. The methods assume that subsurface inflow from the high level water is equal to infiltration into that portion of the drainage basin of the Waianae sector that is underlain by the high level body. Average rainfall over the 25 sq. miles area is about 44 mgd (37 inches per year), the direct runoff at 16 percent of rainfall is 7 mgd, evapotranspiration at 33 percent is 15 mgd, leaving an infiltration residual of 22 mgd. Total groundwater flow, however, is nearly twice as great because of leakage from the Koolau aquifer. Utilizing the Darcy relationship, $Q = TiL$, for the

Waianae aquifer with the following system values,

$h = 20$ feet

$k = 1500$ ft/day

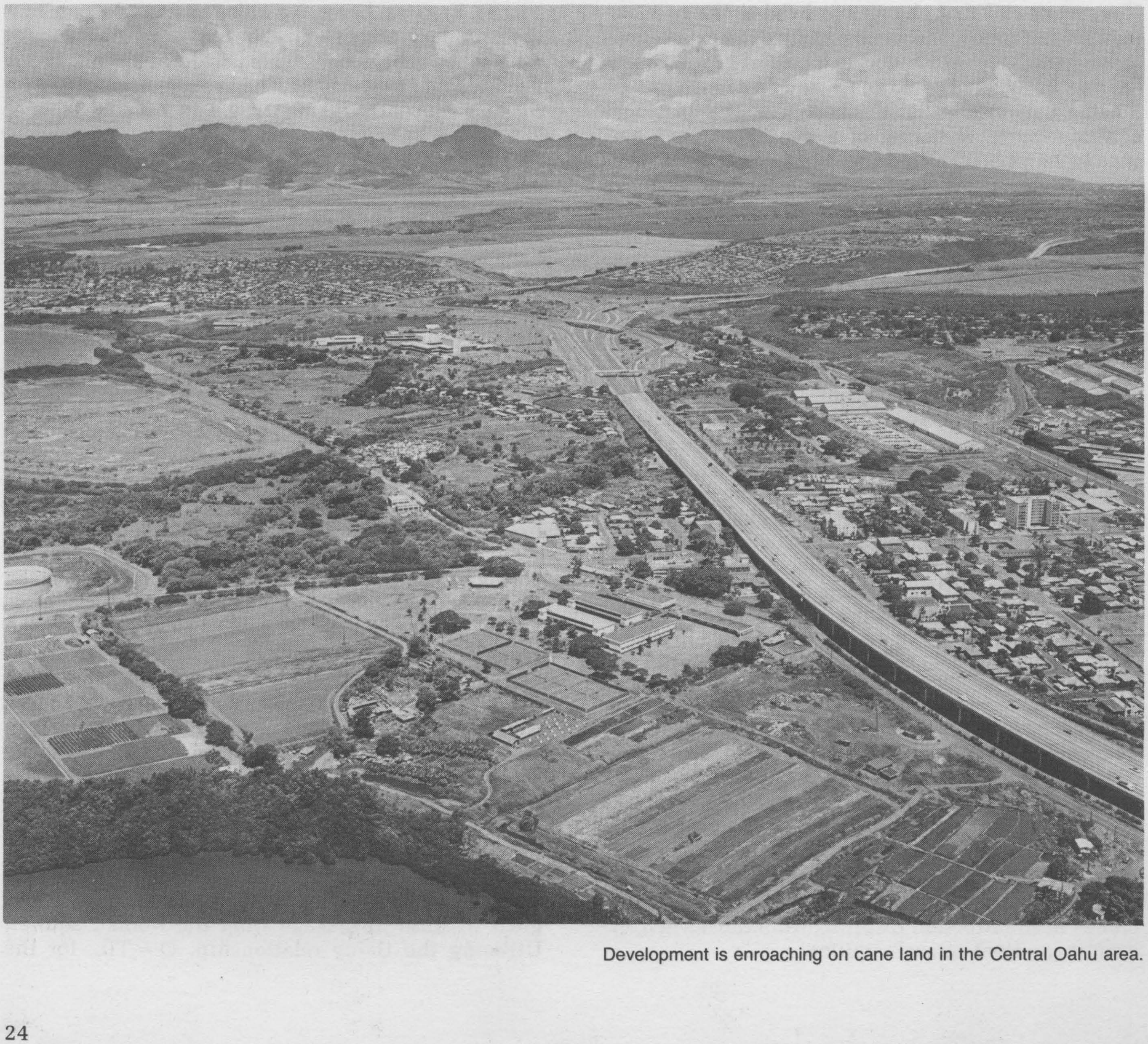
$T = 1.23 \times 10^6$ ft^2/day

$i = 1.3$ ft/mile

$L = 3$ miles

the computed average groundwater flux is 36 mgd. Assigning an average value of 20 mgd to natural rainfall infiltration leaves about 16 mgd for flow across the unconformity, equivalent to 1.78 mgd per linear mile, or 337 gpd/ft. These computations by no means provide exact flow quantities; they merely suggest that the total groundwater flux in the Waianae aquifer averages on the order of 40 mgd, half of which originates as basin rainfall and the other half from the Koolau aquifer. In the discussion of hydrologic budgets for the entire Pearl Harbor aquifer a value of 225 mgd was calculated as the natural infiltration; 20 mgd of this assigned to the Waianae aquifer leaves 205 mgd for the Koolau aquifer.

Draft from the Waianae sector has been averaging 32 mgd for the past decade. Oahu Sugar Company pumps about 29 mgd and the U.S. Navy 2.5 to 3 mgd.



Development is enroaching on cane land in the Central Oahu area.

HISTORY OF DRAFT

Draft in the Pearl Harbor region has been conscientiously compiled on a monthly basis since 1910. This record is of enormous importance in the calibration of models that profess to simulate and predict aquifer behavior. Although draft reported by the principal user of water, the plantations, is obtained by converting the energy consumed in pumping to flow rates, the reported volume is assumed to be accurate enough for mass balance computations. Unless otherwise stated, draft refers to the forcible extraction of water from the basalt aquifers. Draft from the caprock aquifer of the Ewa Plain is not included in the analyses.

Pumping probably started in the Pearl Harbor region not long after the original artesian well was drilled in Honouliuli. Not until 1890 when Ewa Plantation was formed and drilled its first wells, however, did the pumpage become significant. In 1897 the Oahu Sugar Company started well drilling, and the following year the Honolulu Plantation began drilling. These three companies, irrigating from 25,000 to 30,000 acres of cane, were practically the only producers of pumped groundwater in the region until the start of the second world war.

The record of draft since 1910 has been discussed and illustrated by numerous investigators (Stearns and Vaksvik, 1935; Visher and Mink, 1964; Dale, 1967; Soroos and Ewart, 1979) and has been the subject of continuous attention and evaluation by the Board of Water Supply. The meaning of draft frequently differs among investigators. In some cases draft is assumed to equal all outflow from the aquifer, including spring flow, in others it is taken as actual pumpage less a fraction allowed for return irrigation. To avoid confusion the term "draft" should be clearly defined; it should not include any flow that is naturally leaking from the lens near the pumping station, such as springs and seepages, nor should it be modified by a correction for an assumed value of return irrigation, unless qualified. The word draft is strictly defined as only that groundwater that is forcibly extracted from the aquifer. This definition excludes spring flow, pumpage from springs, pumpage from tunnels driven at the site of springs to intercept flow, and all surface water. It is restricted to the lifting to the surface of groundwater that would not otherwise naturally leak or discharge to the surface in the near vicinity of the pump.

For certain hydrological balances, the volume rate of return irrigation flow over the aquifer is subtracted from draft to give net draft. Whenever net draft is used in computations and modeling it should be defined. Normally it is defined as draft minus the sum of return irrigation flows, no matter what the source of the original irrigation water. Thus in the Pearl Harbor region, net draft = draft − return irrigation component of draft − return irrigation component of Waiahole Ditch flow.

For the 20 year interval prior to 1900 essentially no record of draft was kept, and from 1901 to 1910 only partial records are available. According to Wentworth (1951), before 1890 only 10 to 12 wells were drilled in the Pearl Harbor region. These few wells are not likely to have averaged more than five mgd total draft. In 1890-91 Ewa Plantation drilled 20 wells and by the end of the decade had drilled about 50. Oahu Sugar Company and Honolulu Plantation Company added wells to the Ewa completions to give a total of nearly 140 wells by 1900 and 192 by 1910. Based on the staging of well construction as reported by Wentworth and shown in Ewa Plantation records (BWS files) for the period until 1901, and on incomplete pumping records from 1901 to 1910, estimates of total average draft in the Pearl Harbor region prior to the start of good records in 1910 are on Table 3.

TABLE 3

Pearl Harbor Region Draft Estimates: 1880-1910

Period	Total av. Draft mgd
1880-1885	5
1886-1890	5
1891-1895	40
1896-1900	80
1901-1905	90
1906-1909	132

From 1910 until 1940 total draft was comparatively stable. Sugar cane cultivation and processing dominated land and water use and consumed all but a few mgd of the water. Draft varied from year to year as a response to rainfall but within a moderate range. The summary below lists average draft and the minimum and maximum monthly averages for each plantation in this 30 year interval.

TABLE 4

Pearl Harbor Region Draft 1910-1939 (mgd)

Plantation	Average Draft	Average Maximum Month	Average Minimum Month	Av. Max. Month / Av. Draft
Honolulu	44	67	25	1.52
Oahu Sugar	48	77	22	1.60
Ewa	72	97	31	1.35
Total	164	238	78	1.45
Totals by Aquifer				
Koolau	145	210	70	1.45
Waianae	19	28	8	1.47

With the coming of the second world war and the land use disruptions that occurred during and afterwards, the plantations no longer were essentially the sole producers of groundwater. In fact, while total draft increased, plantation draft decreased. Draft by decades since 1940, with statistical moments, is included in the following summary. Statistics are based on annual averages reduced to daily flows.

TABLE 5

Pearl Harbor Region Draft 1940-1978 (mgd)

Period	Total Average Draft	Standard Dev.	Coeff. var.	Plantation Share Draft	Non-Plantation Share Draft
1910-1939	164	16.4	.10	164	<5
1940-1949	170	20.0	.12	150	20
1950-1959	161	19.8	.12	125	36
1960-1969	180	12.4	.07	135	45
1970-1978	216	16.8	.08	135	81

In addition to draft pumped from the basaltic aquifers in the Pearl Harbor region, the plantations used a variety of other sources for irrigation, some of which had to be abandoned as urbanization drove agriculture from south central Oahu. This urbanization, incidentally, did not reduce the acreage in sugar by the number of acres encroached upon; new lands were planted so that the net reduction has been less than 5,000 acres.

The primary extraneous source of water for irrigation since 1916 has been the Waiahole Ditch System. Its availability continues to be an essential element in the success of sugar cultivation and in the hydrologic budget of Southern Oahu.

The Waiahole Ditch System has been providing Oahu Sugar Company with approximately 27 to 33 mgd of fresh water since 1916. It has been a reliable source, as shown by the following statistical data set.

TABLE 6

Waiahole Ditch System Flows (mgd)

Period	Av. Flow	Standard Dev.	Coeff. Var.	Remarks
1916-1926	27.7	3.20	.12	Construction of main tunnel. Start Waikane 1 development tunnel in 1925.
1927-1939	37.7	5.09	.14	Completion of Waikane 1, Waikane 2, Uwau and Kahana development tunnels.
1940-1949	31.0	5.15	.17	System complete.
1950-1959	30.3	2.12	.07	System complete.
1960-1969	32.9	2.66	.08	Extension of Uwau Tunnel by 260 feet.
1970-1978	27.8	2.14	.08	No change in system.

Waiahole Ditch plays a significant role in diverting water from Windward Oahu to the southern slopes of the Waianae range.

A second significant source of non-aquifer water used in irrigation for a long period was diversions from streams. Prior to 1947 Oahu Sugar Company drew a total average of 8.6 mgd from Waimalu, Waiawa, Kipapa and Waikakalaua Streams (Hirashima, 1971). This fell to 5.7 mgd in 1947-1960 and was discontinued as a source shortly after 1960. Even more voluminous was water pumped from the Pearl Harbor springs until the mid 1960's. From Kalauao Springs an average of 2.7 mgd was taken, from Waiawa 1.0 mgd, from Waikele 3.9 mgd, and, starting in 1938, from Hawaiian Electric Company springs and tunnel an average of 7.9 mgd. None of this water except the Waikele component is now diverted; pumping ceased in 1967. Thus the loss of stream water (about 8 mgd) and the springs (about 12 mgd) reduced Oahu Sugar's total supply by 20 mgd just when urban demands on the Pearl Harbor aquifer escalated. Since 1960 several mgd of stream water from lower Waikele was added to their system to partially offset the loss.

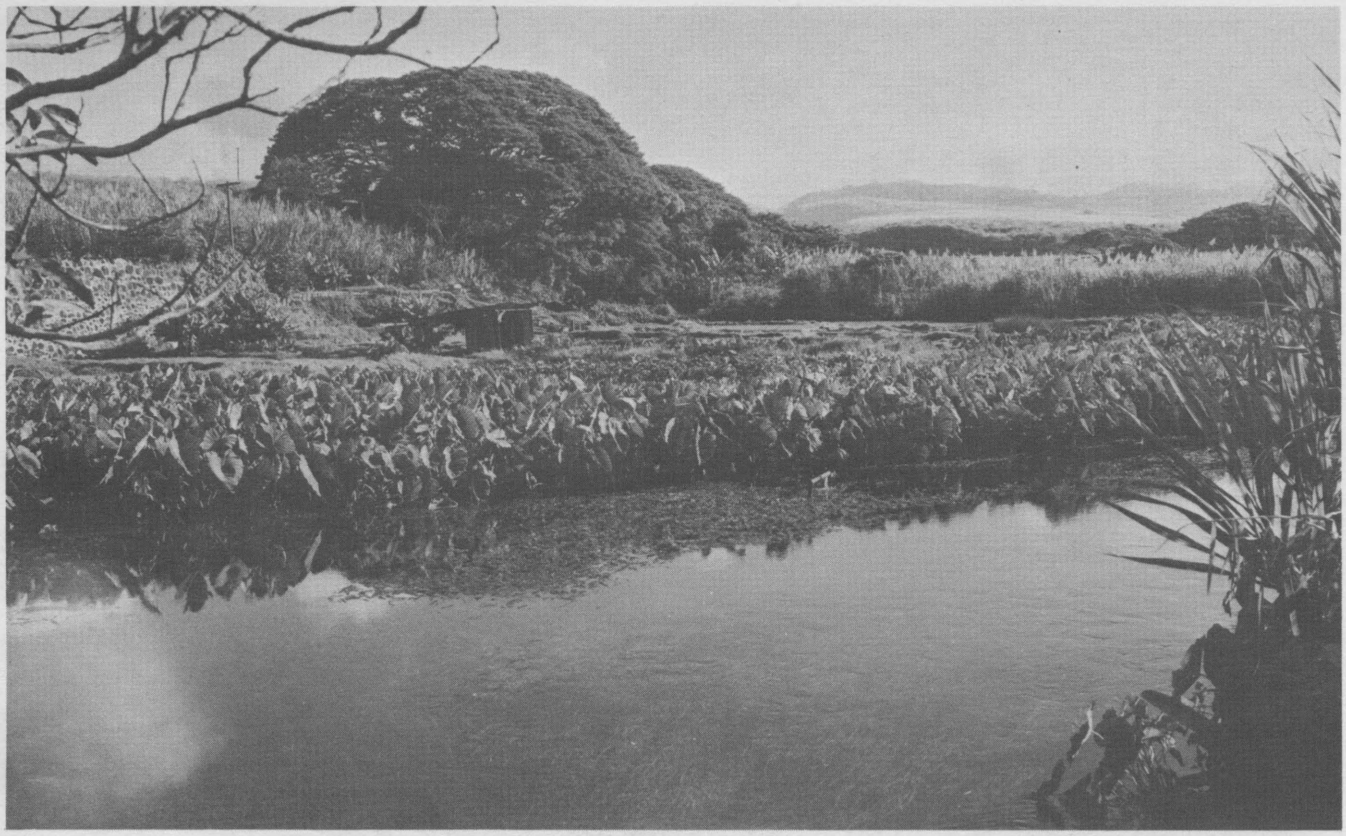
Still another source of non-basalt aquifer water used in irrigation has been the limestone aquifer of the Ewa Plain. Until the last decade an average of about 10 mgd was pumped, but recent draft has been between 20 and 25 mgd. Evidently the loss of stream and spring waters has forced the plantations to exploit more caprock water.

The table below recaps the history of draft in the Pearl Harbor region and of the other sources of water used there. Total draft has increased dramatically in the last decade, while non-aquifer sources (except for the Ewa Plain limestone water) have diminished. Plantation draft, which dominated water usage before 1940, has decreased to an apparently steady level of 135 mgd. In the halcyon era of sugar cultivation in Southern Oahu, plantations used a total of about 220 mgd; today that total has been reduced to about 190 mgd.

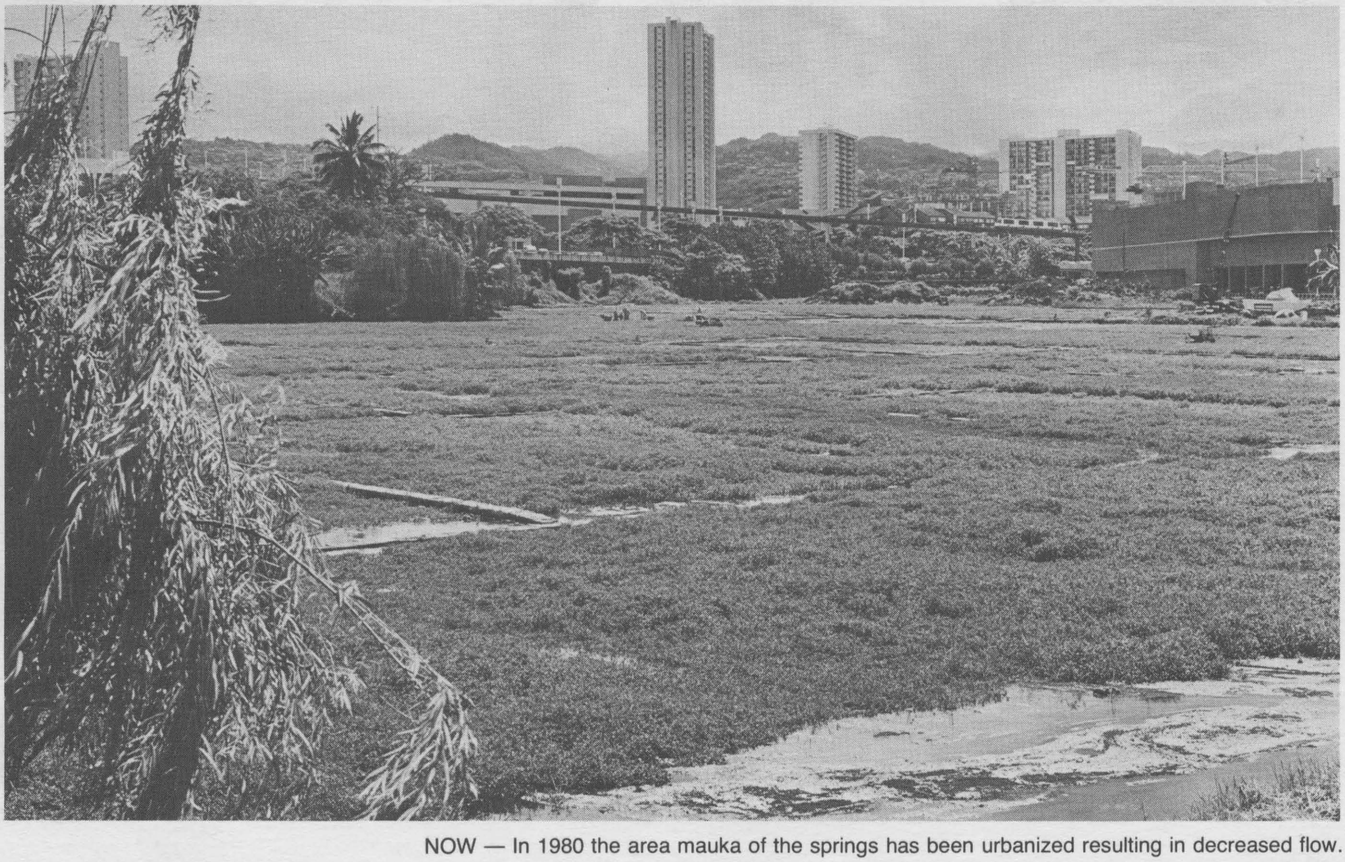
TABLE 7

Pearl Harbor Region. Summary of Water Production (mgd).

Period	Total Draft	Planta-tion Draft	Waiahole Ditch	Streams	HECO and Pearl Harbor Springs	Ewa Lime-stone	Total Planta-tion	Total non-aquifer sources
1920-1939	164	164	35	9	8	5	221	57
1940-1949	170	150	31	7	16	5	209	59
1950-1959	161	125	30	6	16	10	187	62
1960-1969	180	135	33	3	10	12	193	58
1970-1978	216	135	28	4	3	20	190	55



BEFORE — In 1960 Kalauao Springs issued a voluminous flow into Pearl Harbor in the midst of a bucolic setting.



NOW — In 1980 the area mauka of the springs has been urbanized resulting in decreased flow.


One of the free-flowing artesian wells in the Pearl Harbor area used for watercress growing.

HEADS IN SOUTHERN OAHU

Initial Heads

The formulation of an analytical model that describes and predicts behavior of the basal lens over time is markedly simplified if initial head conditions are known. In Southern Oahu there is a precise initial time, the summer of 1879, for the start of the exploitation of the lens, and fortunately some heads were measured nearly at the moment the first wells were drilled. Nevertheless, the assignment of initial heads for the various sub-areas of Honolulu and Pearl Harbor has not been consistent, presumably because the accuracy preferred for non-equilibrium mathematical modeling has not until recently engaged the attention of investigators. Even for Area 2 in Honolulu, where the first well was drilled in 1880 and numerous heads were measured then and in the few years following, the common assignment of 42.0 feet as initial head is not really precise because higher heads were recorded as late as 1889.

In Area 2, the highest reported head was 43.5 feet at Well 38 in March, 1880 (Stearns and Vaksvik, 1938). This well continued to show heads in excess of 42 feet until 1883. Another high head was encountered at Well 52, where a reading of 42.9 feet was made in July, 1882. Well 99 exhibited a head of 42.8 feet in June, 1889, a year when heads in Honolulu had recovered to their original maximums. Evidently, the value of 42 feet as the maximum initial head is too low by as much as a foot and a half. For the robust analytical model used in this study, a compromise initial head of 42.5 feet was selected for Area 2.

Few early heads (pre-1889) were reported for sub-areas of Southern Oahu west of Nuuanu Valley. In Area 3 an initial head of 41.7 feet was given for Well 121, but the earliest measured head reported for Area 4 was in 1898 at Well 149 (head of 29.8 feet), nearly two decades after pumping had started in Honolulu. It was stated (T. F. Sedgewick, 1910, in BWS files) that the initial Area 4 head was 37 feet, but no documentation was given. As will be shown later, an initial head of 37 feet for Area 4 is probably correct, however.

For the Pearl Harbor region apparently no unequivocal record of a measured initial head exists. The earliest head reported by Stearns and Vaksvik (1938) was 31.5 feet for May, 1890, at Well 268 in Honouliuli. This was the year when the first plantation wells were drilled, and so the reported head was likely to be no lower than 0.5 to 1.0 feet of the correct value. An initial head of 32 feet in Honouliuli is reasonable, and, in fact, Thrum's Annual of 1889 (BWS files) stated that the original head at Ewa was 32 feet, a value accepted by Schuyler and Allardt (1889) and subsequent investigators. T. F. Sedgewick (1910, BWS files) suggested that the original Honouliuli head was 33 to 35 feet.

Well 268 is at the lower margin of the hydraulic gradient in the Koolau portion of the basal lens so that its head would be less than at sites further inland. The natural hydraulic gradient in Southern Oahu is one foot per mile, and therefore the most upgradient initial basal head at the Wahiawa high level boundary should have been 39 to 40 feet. At observation Well 244 in Waipahu, the index well used in the analytical model, the comparable initial head would have been 33.5 feet.

The earliest reported head measurements for the central and eastern Pearl Harbor region are for 1910, about 12 years after Oahu Sugar Company and Honolulu Plantation Company had started drilling wells. Heads measured this late after pumping began are too low to be used even as a gross approximation of initial conditions. However, because groundwater flows along a continuous gradient from Area 2 to Area 6 (Pearl Harbor), an estimate of initial head in the eastern Pearl Harbor region can be made by correlating known head behavior among Areas 2, 3, 4 and 6 and referring the correlations to the known initial head of 42.5 feet in Area 2.

For periods of steady draft and nearly constant head, extremely close correlations exist among heads of the various sub-areas. Least squares correlations were computed for 72 steady head-constant draft intervals in the period 1938-1962 for the Honolulu adjacent sub-areas (Area 2-Area 3; Area 3-Area 4) and for 29 intervals in the period 1954-1962 for Area 6-Area 4. The head range for Area 2 was ten feet; for Area 3, 11 feet; for Area 4, 9.5 feet, and for Area 6, 6.8 feet. In all cases the correlation coefficients exceeded .99, about as perfect as attainable for natural systems. The computed original heads, based on an initial head of 42.5 feet in Area 2, were: Area 3, 41.2 feet (the measured initial head was 41.7 feet in April, 1892, as noted earlier); Area 4, 37.0 feet (the same as stated by Sedgewick, see above); and Area 6 (Halawa), 35.1 feet. The computed head for eastern Pearl Harbor is consistent with unsupported statements made in the literature. A further correlation was made between T-45 (Halawa) and T-25 in the midst of the Waiau-Kalauao Springs area. For 27 intervals (1954-1962) the correlation coefficient exceeded .98, and the initial head computed for T-25 was 29.4 feet. This low head undoubtedly is attributable to the rapid increase in hydraulic gradient in the vicinity of the spring discharges.

From the initial heads, the initial gradient from Honolulu to Halawa was calculated to be as follows:

Area 2 to Area 3 — 0.9 feet/mile
Area 3 to Area 4 — 0.9 feet/mile
Area 4 to Area 6 — 1.0 feet/mile

This gradient of 0.9 to 1.0 feet/mile agrees with the gradient in the inland portion of central and western Southern Oahu.

A summary of initial heads for locations in subareas of the basal lens of Southern Oahu is given below:

TABLE 8

Initial Heads, Southern Oahu

Sub-area	Site	Initial head (h_o) ft.	Remarks
Area 2 Honolulu	Beretania	42.5	Original measurement
Area 3 Honolulu	Kalihi	41.5	Original measurement and correlation
Area 4 Honolulu	Moanalua	37.0	Correlation and statements in literature
Area 6 Eastern Pearl Harbor	Halawa	35.0	Correlation and statements in literature
Area 6 Central-Eastern Pearl Harbor	Kalauao-Waiau	29.5	Correlation
Area 6 Central-Western Pearl Harbor	Waipahu	33.5	By gradient from Well 268
Area 6 Western Pearl Harbor	Honouliuli	32.0	Original measurement Well 268
Area 6 Central Pearl Harbor	Waipio	39.0	By gradient from Well 268

Storage and Operating Heads in the Pearl Harbor Region

Groundwater heads in Southern Oahu are measured as the aquifer is being pumped and thus these heads reflect pumping stresses as well as the storage state of the groundwater system. The difference between maximum and minimum measured heads over a short time interval, such as a year or less, is predominantly the result of transient behavior generated by changes in pumpage. The volume of groundwater in the lens diminishes until equilibrium is achieved, but the proportion lost over a year is small in a large aquifer. In the Pearl Harbor region head loss due to volume reduction has averaged less than .25 feet/yr. since 1910, while water level changes of up to ten feet seasonally are caused by pumping.

In analyzing storage and yield states of a basal lens it is crucial that measured heads are not taken as storage heads, yet this assumption is implicit in every evaluation that has been made of the Pearl Harbor aquifer. Two categories of head need to be defined to avoid confusion. The head that is measured in the normal manner by tape or airline while the aquifer is being pumped is the "operating head"; the head that expresses the vertical dimension of fresh water in the lens is the "storage head." The operating head gives information about the response of the upper surface of the lens to pumping, while the storage head indicates the volume status of the lens. Unfortunately the storage head cannot be simply directly measured, but occasionally when draft is severely reduced, as it used to happen in the Pearl Harbor region during wet winters, the measured head recovers toward the storage head.

Depending on location in the Pearl Harbor region, operating head may vary over a range of ten feet or so annually. The reasons are not hard to find; draft is extremely large and the aquifer is bounded. The dimensions of the Pearl Harbor portion of the Southern Oahu aquifer is ten miles in a north-south direction and 20 miles east-west. It is a large aquifer but is neither infinite nor so extensive that the groundwater hydraulic equations, which assume no boundaries, are serenely applicable. Dewatering of the top of the aquifer in response to pumping is substantial.

The basal lens is so large in its vertical dimension that its bottom can respond only very slowly to pumping stresses. Wentworth recognized this tendency and developed the concept of bottom storage to account for it. He minimized the role played by natural leakage in discharging bottom storage over long time periods and concluded that most storage had been extracted by pumping. Actually most bottom storage leaves the system as leakage, and the time over which this leakage occurs is the transient period of head decay.

The bottom storage concept, amended to include leakage as the principal discharge parameter, qualitatively explains why the water table of the lens varies so greatly with draft. If there were no bottom storage, the lower surface of the lens would respond instantaneously to changes in the top surface. All evidence shows that this does not happen. For the time intervals in which seasonal variations in pumpage occur, the base of the lens acts as if it were a stable lower boundary. The volume of water that either must enter or leave bottom storage is far too great to sustain the Ghyben-Herzberg equilibrium. For instance, a five feet seasonal change in head in the Pearl Harbor region, which is common, would require the movement of about 1,000 mgd into or out of the lens to sustain the Ghyben-Herzberg ratio of 40 to 1. This enormous rate of flow is about four times the natural daily recharge. Neither leakage nor pumpage could account for it.

Of course, the bottom of the lens in Southern Oahu contracts or expands over the long term but at a rate that is very small relative to annual changes of the water table. This transient behavior of the lower surface is overwhelmed by the large head fluctuations caused by pumping.

The magnitude of seasonal changes in operating heads can be deduced from hydraulic theory. Taking the boundaries of the Pearl Harbor aquifer as the Koolau rift zone on the east, the Wahiawa high level water on the north, the caprock wedge on the south, and the Koolau-Waianae unconformity on the west, and assuming that the bottom of the lens acts as an immobile lower bound, image theory can be applied to compute expected drawdown at different observation wells. The lateral boundaries are treated as impermeable in view of the fact that head lowering does not induce appreciable additional recharge. Only one image well for each boundary for each pumping center is employed because additional images would be so distant as to produce vanishingly small increases in drawdown.

The period of time for the analysis is 1910-1935 when essentially all of the pumpage in the Pearl Harbor region was for sugar cane irrigation. Pumpage was restricted to five centers, as follows on Table 9.

TABLE 9

Average Draft Pearl Harbor Region, 1910-1935

Pumping Center	Wells	Average draft, mgd	Percent of total draft
Halawa	185, 186, 189	25.0	18
Waimalu	196, 197	17.8	13
Waiawa	239	10.5	8
Waipahu	246, 247, 248, 249, 254, 4B	31.0	22
Ewa	257, 259, 263, 264, 268, 273	53.5	39
	TOTAL	138	

Computations were made for average drawdown at standard observation Well 193 (Waimalu), 201 (Pearl City), 244 (Waipahu), and 266 (Ewa).

Image theory employs the basic equation of hydraulic flow in porous media to a vertical line sink (pumping well):

$$(1)\quad s = \frac{Q}{4\pi T}\left\{w(u) + w(u)_i + w(u)_{i+1} + \ldots w(u)_n\right\}$$

where s is drawdown, Q is constant pumpage, T is transmissivity, and w(u) is the well function. In this function, $u = \frac{r^2S}{4Tt}$, in which r is distance from the pumping center, S is specific yield, and t is time.

The function w(u) pertains to drawdown directly induced by the pumping well; the functions $w(u)_n$ pertain to drawdowns induced by image wells located across the boundaries from each pumping center. In the calculations T is taken as 12 mgd/ft., S as .10, Q as 138 mgd, and t as 200 days (to extend over the pumping season).

For the period 1910-1935 the average drawdown experienced each year by dewatering of the upper portion of the lens as a result of pumping is computed to have been as follows on Table 10.

In equation (1) Q only is a linear constant, and therefore the expected operating drawdown at a given site for any value of Q can be obtained by the simple relationship, s (total) = Q $(\overline{s}/\overline{D})$. In a recent publication by the USGS (Soroos and Ewart, 1979), average annual operating heads of several wells, including Well 244, are plotted for the period since 1910. The average heads plus the average drawdowns given in the table above correspond with the storage heads computed by the robust analytical model. For instance, the operating head of Well 244 in 1935 was 21 feet whereas the computed storage head was 26.0 feet, a difference of five feet comparable to the drawdown of 6.3 feet given in the table.

Even though the pumping centers in the present time are not arranged as they were in the plantation era, the drawdown per mgd pumped can still be reasonably applied to modern conditions. The current high average rates of regional draft (about 200 to 230 mgd) could be expected to cause drawdowns of approximately nine feet at Well 244 and ten feet at Well 266. Summer time draft would increase these values by several feet.

TABLE 10

Pearl Harbor Region: Drawdown Caused by Pumping

Observation Well	Drawdown from pumping wells ft.	Drawdown from image wells ft.	Total drawdown ft.	Ratio total drawdown to total draft
193	2.4	1.4	3.8	.0270
201	2.5	2.1	4.6	.0333
244	3.4	2.9	6.3	.0457
266	3.2	3.6	6.8	.0493

History of Heads in the Pearl Harbor Region

Initial heads may not have been measured for all but the Honouliuli sector of the Pearl Harbor area, but since the turn of the century a good record has been kept for wells and observation sites in an approximately one mile wide band near the harbor. Heads in this zone often appear confusing, however, because they are affected by pumping, which is concentrated near the harbor, and by the large increase in hydraulic gradient just inland of the Pearl Harbor springs. Not until the 1950's were sites available for head measurements in the interior portion of the region, away from the pronounced effects of pumping and spring flow.

The most interpretable of the measured heads are those taken when pumping ceased or was greatly diminished at sites beyond the major hydraulic effects of the springs. These are typically the maximum annual heads and, as explained earlier, approach true storage heads for pumping shutdowns of several months.

Nevertheless, in the analysis of the behavior of the basal lens measured heads have always been assumed to be identical to storage heads and have traditionally been reported as annual or moving averages. Only the trend line of the average is meaningful because it indicates the rate at which storage is depleted. A truer picture of the storage state of the basal system is given by maximum heads.

Since 1910 there have been seven periods during which heads in the Pearl Harbor region so strongly recovered as to approach true storage heads. These periods were 1916-18, 1927-28, 1932-33, 1937-38, 1951-52, 1956-57 and 1968-69. Rainfall evidently was so plentiful and well distributed in the winter months that plantation draft for irrigation was unnecessary, allowing recovery to proceed over several months. As recently as 1968-69, Well 244 at Waipahu had recovered to 25.2 feet. The table below summarizes maximum heads since 1910 for sites near the harbor, for inland sites, and for wells in the Waianae aquifer.

A water table contour map of maximum heads at times of equilibrium is the most accurate portrayal of the state of the system determinable from the records. Maps based on average heads underspecify the storage condition of the lens. In particular, they do not show equipotential distributions since measured heads never express true potential. The analytical model suggests that the only period of equilibrium in the Pearl Harbor region since 1916 was for about seven years between 1946 and 1952; near equilibrium persisted until about 1959. From 1916 to 1945 the lens shrank as it adjusted to the recharge-draft-leakage conditions. Shrinkage resumed in 1959 and has been continuous since then.

Figure 2 is a water table contour map of the original equilibrium condition to fit initial heads of 32 feet at Well 268, 35 feet in Halawa, and 39 feet at the

TABLE 11

Maximum Operating Heads (ft.), Pearl Harbor Region 1910-1979

Location	Initial Head	1916-1918	1927-1928	1932-1933	1937-1938	1951-1952	1956-1957	1968-1969	1979-1980
Sites near harbor, Koolau aquifer									
Halawa	35		26.0	25.5	24.0		24.1	22.5	17.1
Waimalu	30		25.8	24.4	24.0		23.6	23.2	17.2
Pearl City	34	31.2	25.6	22.8	23.5			21.4	15.4
Waipahu	33.5	30.0	26.4	25.3	25.9	24.4	24.8	25.2	19.2
Honouliuli	32	29.2	27.0	25.4	25.5		24.5	25.9	19.2
Inland sites, Koolau aquifer									
Pearl City			27.8	27.1	27.5		27.0		
Waipahu			27.3	27.2			26.0		
Waipio	39						31.2		
Waianae aquifer									
Well 274			19.7	20.0					
Well 276		16.4	15.4	14.5			14.0		

Wahiawa high level boundary. Head at the inland boundary is based on a gradient of one foot per mile, which is consistent with the basic parabolic equation of flow in the basal lens,

$$(2) \quad q = \frac{41k\,h^2}{2x}$$

where q is flow through the full thickness of the aquifer per unit width, k is hydraulic conductivity, h is head, and x is theoretical distance from h = o to h. For the Pearl Harbor region, letting Q = 225 mgd gives a value for q of 407 ft³/day/ft, and thus for k = 1,500 ft/day the theoretical distance, x, to Well 268 would be 77,366 feet. Transformation of equation (2) employing the distance, L, between Well 268 and the Wahiawa high level boundary gives:

$$(3) \quad h_L^2 = \frac{h^2(x+L)}{x}$$

where h_L is head at the inland boundary. For the values given above, $h_L = 38.9$ feet, resulting in an average gradient of one foot per mile. The water table parabola is so gentle as to be practically straight.

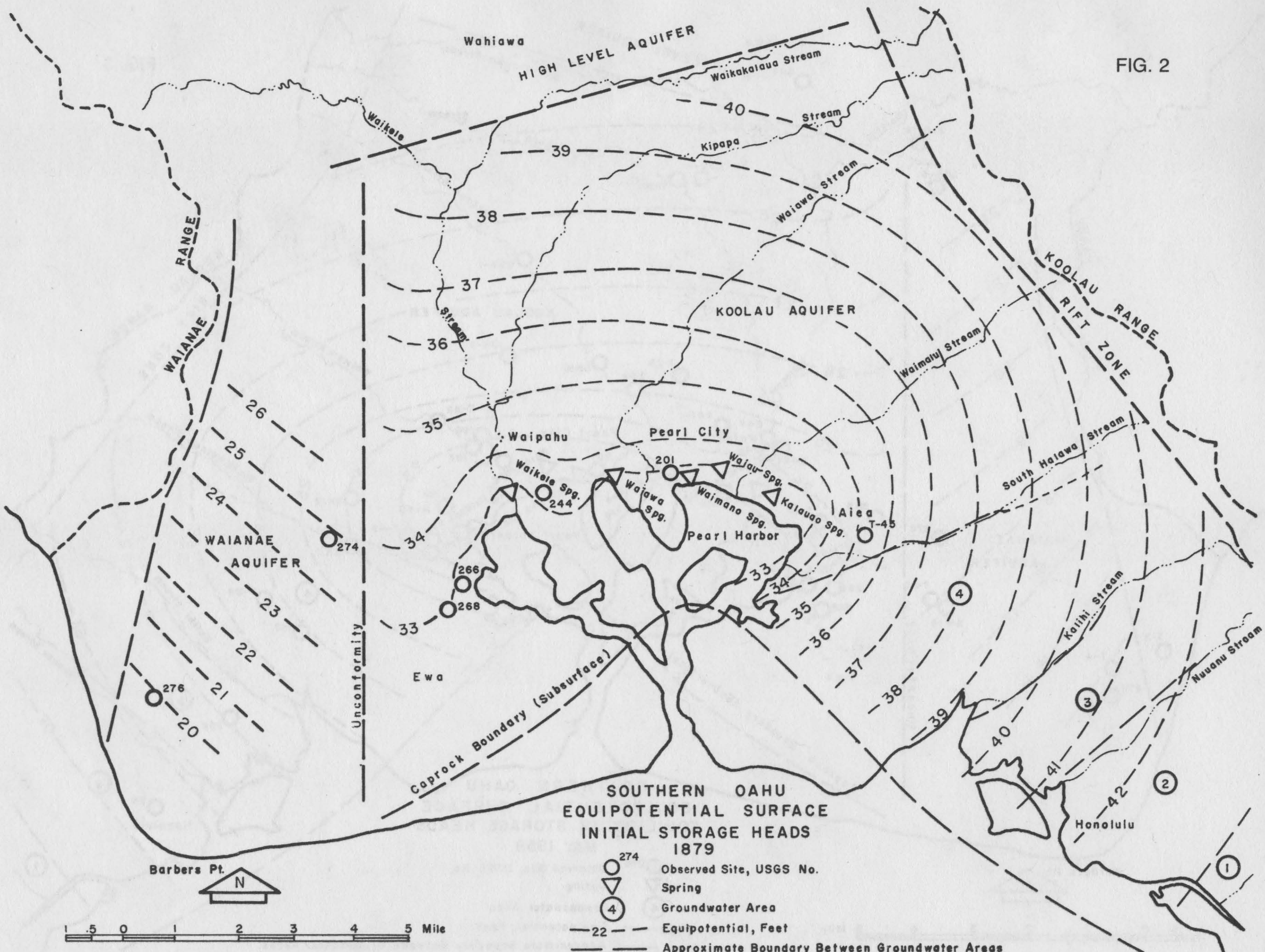
Storage head water table contours for the quasi-equilibrium period in 1958 are shown in figure 3 which is based on the recovery data of the sugar strike in addition to the satisfaction of theoretical considerations. Because the water table is parabolic, it is expressed in the simple equation:

$$(4) \quad h_d = h_u - b\,x^{1/2}$$

in which h_d is a down gradient head, h_u is an upgradient head, x is the distance between them, and b is a constant. For the maximum recovery heads, in particular for observation holes T-47 and T-29, the value of b is .01884. The theoretical head at the Wahiawa boundary (Well 250-2) was 29.3 feet; the actual measured head was 29.1 feet. The theoretical head for Well 239-1 was 27.5 feet; actual measured head was 27.7.

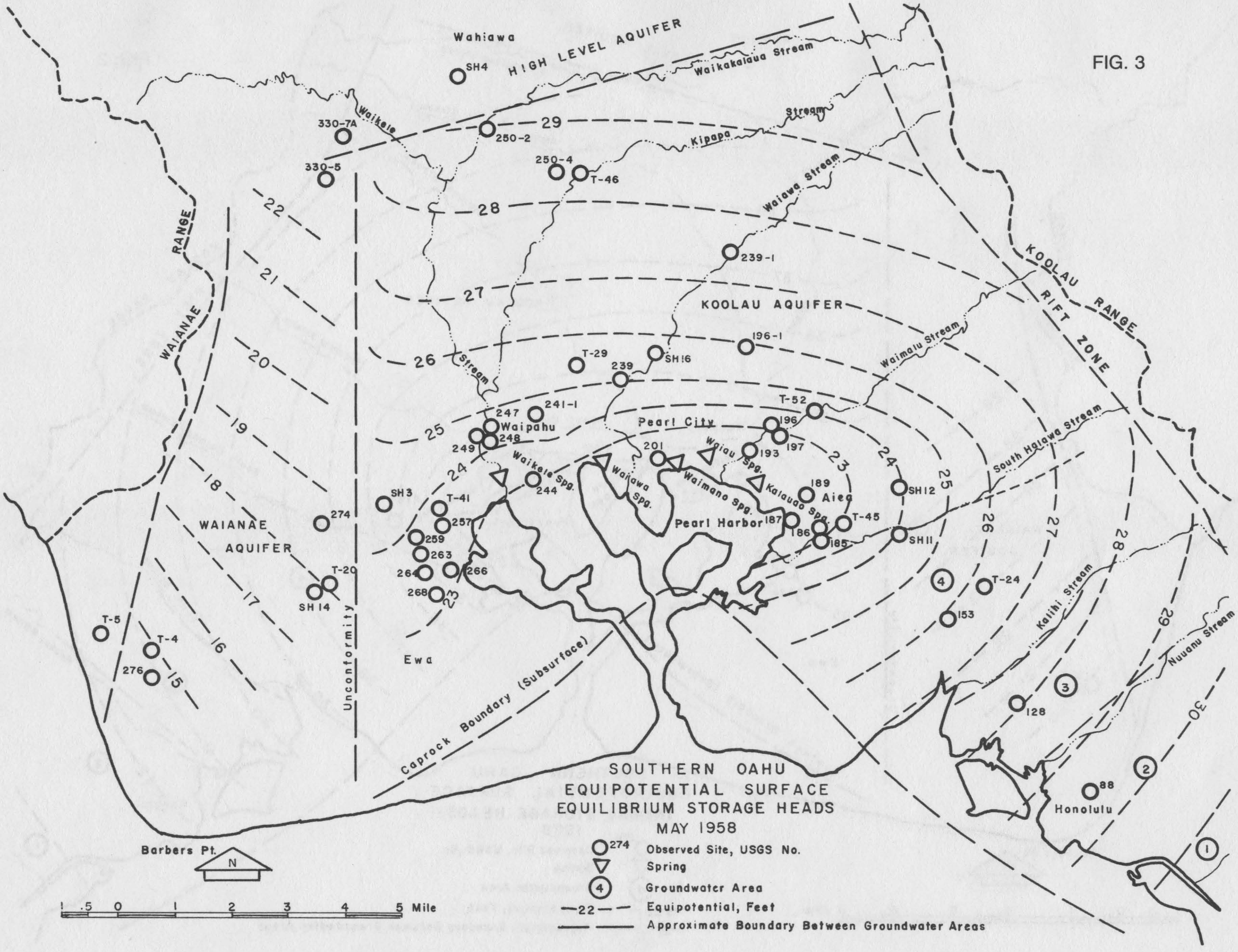
Figure 4 maps the approximate current storage heads. The system is not in equilibrium, and thus the heads reflect a transient phase. Had the initial conditions not been disturbed, or had the draft and water use pattern of 1958 not been altered, storage heads for those periods would not have decayed further. On the other hand, present heads will continue to fall until an equilibrium is reached.

FIG. 2



Wahiawa
HIGH LEVEL AQUIFER
Waikakalaua Stream
Waikele
40
39
38
37
36
35
34
33
Kipapa
Stream
Waiawa Stream
Stream
KOOLAU AQUIFER
KOOLAU RIFT ZONE
KOOLAU RANGE
WAIANAE RANGE
Waimalu Stream
South Halawa Stream
26
25
24
23
22
21
20
WAIANAE AQUIFER
Unconformity
Waipahu
Pearl City
Waikele Spg.
201
Waiau Spg.
Waiawa Spg.
Waimano Spg.
Kalauao Spg.
Aiea
T-45
244
274
266
268
276
Pearl Harbor
Ewa
Caprock Boundary (Subsurface)
Kalihi Stream
Nuuanu Stream
41
42
Honolulu
Barbers Pt.
N
1
2
3
4
SOUTHERN OAHU
EQUIPOTENTIAL SURFACE
INITIAL STORAGE HEADS
1879
274 Observed Site, USGS No.
Spring
4 Groundwater Area
22 Equipotential, Feet
Approximate Boundary Between Groundwater Areas
1 .5 0 1 2 3 4 5 Mile

FIG. 3


SOUTHERN OAHU
EQUIPOTENTIAL SURFACE
EQUILIBRIUM STORAGE HEADS
MAY 1958
274 Observed Site, USGS No.
Spring
4 Groundwater Area
22 Equipotential, Feet
Approximate Boundary Between Groundwater Areas
1 .5 0 1 2 3 4 5 Mile
N
Barbers Pt.
Wahiawa
HIGH LEVEL AQUIFER
KOOLAU AQUIFER
WAIANAE AQUIFER
WAIANAE RANGE
KOOLAU RANGE
KOOLAU RIFT ZONE
Unconformity
Caprock Boundary (Subsurface)
Waikakalaua Stream
Kipapa Stream
Waiawa Stream
Waikele Stream
Waimalu Stream
South Halawa Stream
Kalihi Stream
Nuuanu Stream
Waipahu
Pearl City
Aiea
Pearl Harbor
Ewa
Honolulu
Waikele Spg.
Waiawa Spg.
Waiau Spg.
Waimano Spg.
Kalauao Spg.
SH4
330-7A
330-5
250-2
250-4
T-46
239-I
196-I
SH16
T-29
239
241-I
247
248
249
T-52
196
197
193
201
189
SH12
SH11
T-45
187
186
185
244
SH3
T-41
257
259
263
264
266
268
274
T-20
SH 14
T-5
T-4
276
T-24
153
128
88
1
2
3
4
15
16
17
18
19
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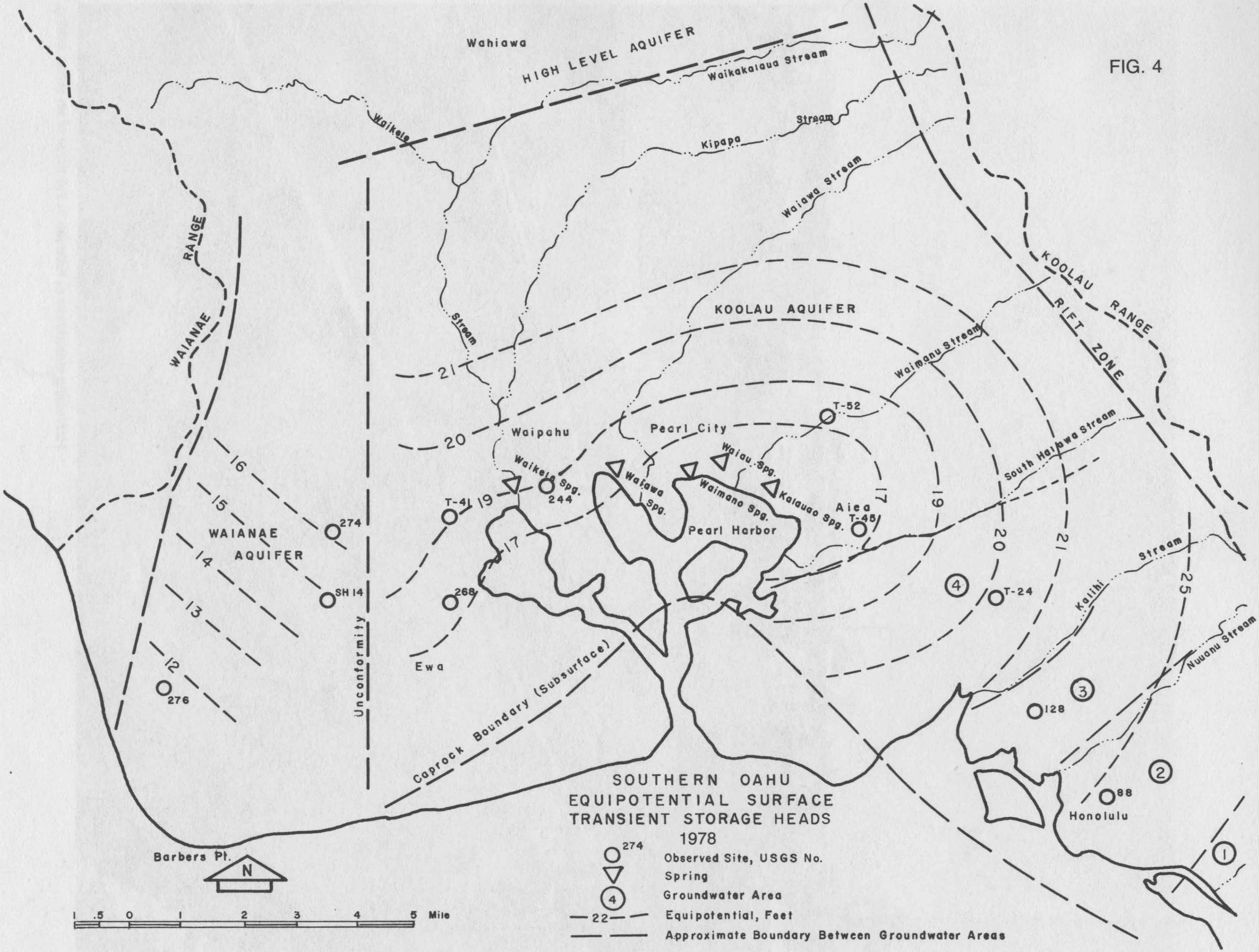
21
22
23
24
25
26
27
28
29
30

FIG. 4



Wahiawa
HIGH LEVEL AQUIFER
Waikakalaua Stream
Waikele
Kipapa
Stream
Waiawa Stream
Stream
WAIANAE
RANGE
KOOLAU
RANGE
RIFT
ZONE
KOOLAU AQUIFER
Waimanu Stream
South Halawa Stream
Kalihi
Stream
Nuuanu Stream
Unconformity
Waipahu
Pearl City
Waikele Spg.
Waiawa Spg.
Waiau Spg.
Waimano Spg.
Kalauao Spg.
Aiea
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T-45
T-24
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SOUTHERN OAHU
EQUIPOTENTIAL SURFACE
TRANSIENT STORAGE HEADS
1978
274 Observed Site, USGS No.
Spring
4 Groundwater Area
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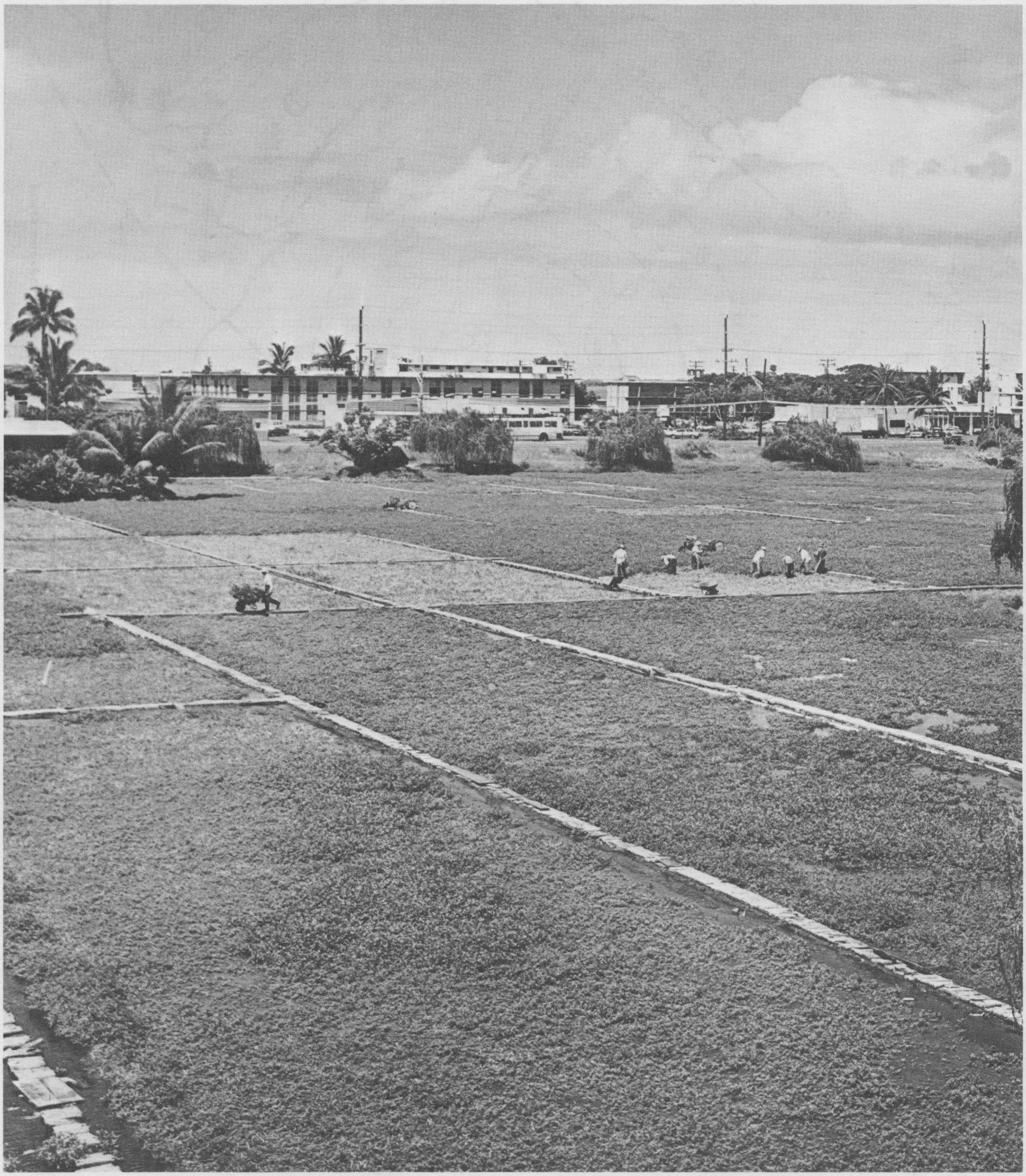
Approximate Boundary Between Groundwater Areas
1 .5 0 1 2 3 4 5 Mile



Harvesting watercress at Kalauao in the midst of an urban setting.

GROUNDWATER LEAKAGE

The basal aquifers of Southern Oahu are unusually voluminous because the caprock sediments of the coastal plain and valley mouths severely inhibit the passage of groundwater from the permeable basalt into them, creating a buildup of potential in the lens great enough to force flow toward discharge sites at elevations higher than the margin of the impeding members of the caprock. Before exploitation commenced in 1879, all flow had to leak from the system to balance recharge. Heads as high as 43 feet in Honolulu and nearly 40 feet several miles inland of Pearl Harbor were required to strike the balance.

At initial conditions in the Honolulu District west of Manoa Valley an average of about 60 mgd had to discharge. Some of it overflowed the inner boundary of the caprock to form wet lands on the coastal plain, a small portion seeped into the caprock, and the remainder flowed westward toward the discharge sink of the Pearl Harbor springs. The common attribution of flow escaping at the toe of the caprock wedge is not a realistic model. In the Pearl Harbor region much of the original flow of about 225 mgd discharged at the arc of large springs from Kalauao on the east to Honouliuli on the west, but a significant portion also seeped into the upper limestone strata in the caprock of the Ewa Plain at its inner margin below the blanket of coarse alluvial sediments. A much smaller amount passed into the thicker caprock wedge. These patterns of discharge, in Honolulu and Pearl Harbor, persist today.

The Pearl Harbor springs are a dramatic manifestation of leakage from the basal aquifer, but measurements made of them fall short of giving total aquifer leakage. As long ago as 1889 Schuyler and Allardt measured flows at the main spring wasteways and arrived at a total flow of 75.5 mgd distributed as follows on Table 12.

Schuyler and Allardt noted that the above was unused spring water and included only the largest streams that could be measured. They stated that the springs irrigated about 2,000 acres of rice fields in addition to banana and other water loving plants. The largest and strongest streams were reported to have their origins at elevations of 20 to 25 feet, consistent with the regional head of the time.

TABLE 12

Pearl Harbor Springs Flow, 1889 (Schuyler and Allardt)

Site	Spring Area	Flow (mgd)
Ah In's Rice Mill (boat channel)	Kalauao	18.0
Aki's Rice Mill	Waiau	6.7
Puikani	Waimano	8.7
Waiawa	Waiawa	14.6
Waikele	Waikele	27.5
	Total	75.5

Four decades later, measured flows at the major springs as reported by Stearns (1931) and Stearns and Vaksvik (1935) were as follows:

TABLE 13

Pearl Harbor Springs Flow 1928-1933 (Stearns, 1931, and Stearns and Vaksvik, 1935)

Spring Area	1928 av. flow mgd	1932 av. flow mgd	1933 av. flow mgd
Kalauao	19.4	21.7	20.9
Waiau	9.0	8.7	8.3
Waimano	11.7	28.7	27.3
Waimano (misc.)	—	3.0 (est.)	3.0 (est.)
Waiawa	16.2	15.2	14.6
Waikele	9.8	8.0 (est.)	8.0 (est.)
Total	66.1	85.3	82.1

In 1938 Hawaiian Electric Company altered flow characteristics of the Waimano Springs complex by driving a tunnel to intercept groundwater, but the total discharge of the tunnel and the springs remained the same as before. The USGS established continuous flow measurement stations at the main springs. In the Visher and Mink study (1964) a linear correlation equation based on the USGS measurements was used to compute average flows for the major springs as follows:

TABLE 14

Pearl Harbor Springs Flow 1953-1957 (Visher and Mink, 1964)

Spring Area	Flow mgd
Kalauao	19
Waiau-Waimano	32
Waiawa	14
Waikele	22
Total	87

The increase in total discharge from Schuyler and Allardt's 75.5 mgd to Stearns' 85 mgd to Visher and Mink's 87 mgd reflects more thorough measurements rather than truly increased outflow because flow apparently was rising as regional head decreased, an impossible relationship.

The USGS initiated a new program in 1967 following changes in the spring areas brought about by urbanization. Flows are now measured several times a year rather than continuously. Evidently a much more extensive network of sites is measured than previously. In February, 1968, the total recorded flow, including Waikele Springs, was 73.4 mgd. Results since 1973, given as totals for the entire network and separately for Kalauao, Waiawa and the Hawaiian Electric Company tunnel, are as follows (data supplied by USGS):

TABLE 15

Pearl Harbor Springs Flow 1973-1979 (USGS Data)

Date	Total flow mgd	Kalauao mgd	Hawaiian Electric Tunnel mgd	Waiawa mgd
6/29/73	51.8	8.85	8.10	8.98
9/25/73	46.7	7.36	8.53	9.18
3/28/74	78.0	11.8	9.37	14.1
3/ 3/74	80.4	12.9	9.57	17.2
6/ 9/75	70.6	12.2	8.98	13.8
1/23/76	69.0	11.2	9.57	12.8
6/10/76	55.4	9.82	7.95	10.5
1/11/77	57.2	9.63	8.60	11.0
4/ 6/77	44.8	7.75	7.37	9.63
9/12/77	40.3	7.75	6.85	8.92
3/29/78	42.2	8.66	6.98	9.95
8/17/78	42.0	7.50	6.98	9.31
4/12/79	67.8	10.2	9.75	13.6
9/ 5/79	54.9	9.37	8.98	11.3

The Kalauao Springs are the best indicator of change over time because the channel in which measurements are made has been about the same since the time of the first USGS measurements. The average flow of 19 to 22 mgd that persisted into the 1960's has been halved as a result of head reduction.

Table 15 illustrates that spring flow is a function of head; when draft is heavy in the summer and autumn months, head falls and discharge is lowered. The relationship between flow and head plots nicely as a straight line on rectilinear paper, so it is used to estimate local and total flow when only head data is available. The relationship between flow and head is not linear, however, but quadratic. It is a coincidence that the straight line correlation follows the trace of the correct parabolic curve in the head-discharge range experienced.

The quadratic characteristic of the relationship is easily demonstrated. The depth of flow, Z, in Darcy's Law,

(1) $$q = -kZ \frac{dh}{dx}$$

is equal to h (1+b) in which b is the density of fresh water divided by the difference in density between salt and fresh water. The specific flux is q, the hydraulic conductivity is k, and the gradient is dh/dx. Restated, the equation is,

(2) $$q = -k\,(1+b)\,h \frac{dh}{dx}$$

which for the limits x (0, x) and h (h,0) becomes,

(3) $$q = \frac{k\,(1+b)\,h^2}{2x}$$

For the fresh-sea water system, b = 40, thus,

(4) $$q = \frac{41kh^2}{2x}$$

or by combining constant terms into the constant, c,

(5) $q = ch^2$

Only a quadratic correlation is valid over the full range of head data. Correlation of the 1973-1979 discharge data for Kalauao Springs with the simultaneous head at Well 187-B using both the linear and quadratic relationships clearly illustrates the interpretability of each. The quadratic correlation gives the following expression,

(6) $Q = 2.00 + .0310h^2$

for which .97 is the correlation coefficient. The linear correlation is,

(7) $Q = -5.78 + .9893h$

for which .97 is also the correlation coefficient. The intercept of equation (6) implies that when head becomes zero, flow continues at 2 mgd, an impossibility, but the excess of 2 mgd from the expected intercept of zero is attributable to a limited data set and normal errors of measurement. For the linear correlation (equation 5) the intercept at zero head is negative; flow supposedly would cease when head fell to 5.8

feet. Provided seepage outlets are available, hydraulically it is impossible to have zero discharge at positive head. Figure 5 is a plot of the quadratic and linear curves obtained by least squares correlation showing their coincidence in the data range experienced. The linear correlation predicts less flow at the initial head of 30 feet than the quadratic correlation (24 mgd versus 31 mgd) and smaller flows at very low heads.

The above hydraulic analysis ignores the effect of the position of the discharge sites and the geometry of the individual orifices. A precise expression is not likely to be attainable in view of the extraneous variables that complicate the simple hydraulic equation. The linear correlation appears to be adequate for its data range but should not be extrapolated beyond.

In the analytical model that simulates the past history of the groundwater system and predicts future behavior under different constraints, leakage does not appear directly in the equations. It is incorporated in the head term, from which it can be computed for any time. Computed leakage is always greater than the sum of the discharges from the springs because it includes unmeasured seepage into caprock as well as into spring areas not incorporated in the USGS network.

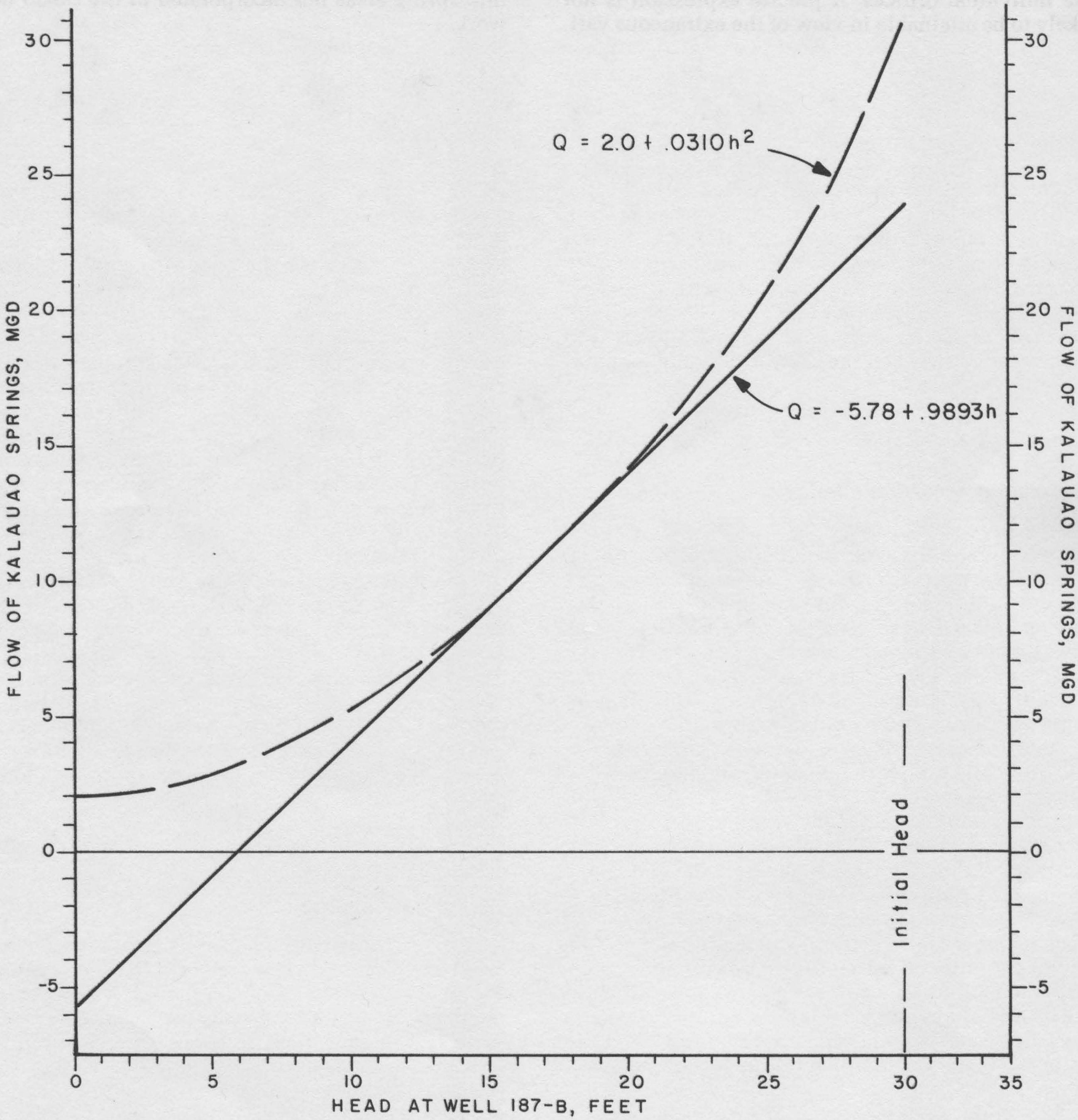
FIG. 5

KALAUAO SPRINGS

FLOW RELATED TO HEAD WELL 187 B

PERIOD: 6-29-73 to 9-5-79; 14 MEASUREMENTS

DATA: U.S.G.S.



BEHAVIOR OF THE AQUIFERS OF SOUTHERN OAHU SIMULATED BY AN ANALYTICAL MODEL

Once the investigation got underway it became evident that the traditional methods employed in analyzing the hydrology of Southern Oahu did not need to be repeated because the limits of their effectiveness had nearly been reached. A different approach was called for, one that focused on transient behavior of the system in a coherent analytical framework rather than on descriptions and assumptions of steady state.

Modeling groundwater behavior of the lens

The very word "modeling" conjures an abstract, often incomprehensive means of describing a system and how it behaves, and too often this is the case, not infrequently by design. On the other hand, without perhaps having been aware of it at the time, most investigators of the water resources of Southern Oahu have modeled the dynamic and static features of a basal lens in pursuit of explaining its behavior. Wentworth invented a complicated model based on bottom storage, another on long term effects of rainfall, and still another that ingeniously describes dispersion phenomena. In contrast, Stearns dealt with the basal lens in a rather fragmented way but nevertheless stressed the interconnection of all of Southern Oahu. The USGS has been modeling the region for a long time. One of the first efforts of the Water Resources Research Center at the University of Hawaii was to construct physical models of a simple Ghyben-Herzberg lens; subsequently they have moved on to more elegant techniques. Individuals at the Board have begun to view all of Southern Oahu as a single resource, implying the interrelation of its parts and therefore its susceptibility to regional modeling.

Modeling need not be complicated to yield a realistic understanding of the groundwater system. The most straight-forward and intellectually palpable models are physical ones of the sand box or Hele-Shaw types. What is seen and measured in these models suggests general behavior and may verify singular phenomena, but these observations could not likely be employed in specific managerial decisions. Word models, such as those of Wentworth, produce deep insights but because of their qualitatism are difficult to translate into the planning process. The next level, mathematical models, evades the ambiguity of words by defining behavior according to a set of precise rules (governing equations) to be applied within a defined space (boundary conditions) following a known equilibrium state of the system (initial conditions). Mathematical models described with linear partial differential equations use analytic solutions; those based on non-linear equations use finite difference or finite element methods that require considerable computer time. Mathematical models are not foolproof; if the boundary conditions and internal parameters of the system are poorly known, as is often the case and, surprisingly, is true for Southern Oahu, sophisticated models may give poor results. Sophistication by no means guarantees accuracy. For this study the model being developed is essentially analytical and can be handled with a programmable calculator, though a computer would expand its utility.

Goals of the model

The dependencies among the major components in the basal groundwater system of Southern Oahu is the fundamental basis of the model. These components are storage volume, storage head, infiltration, draft, leakage and time; they are interrelated by the continuity equation combined with Darcy's Law. The primary object of analysis is to obtain storage head at a point as a function of time rather than to solve for head distribution, the normal objective of mathematical models. Because of the large hydraulic conductivity of the Koolau and Waianae basalt aquifers, spatial distribution of head is easily approximated from head at a carefully selected point. A model presuming to solve for head distribution would require a considerably more sophisticated finite mathematics approach than the storage model and would also require a more accurate knowledge of the boundary conditions than is now available. Indeed, at this stage a head distribution model is more likely to be misleading than instructive.

The model has two principal goals, first to determine the present storage state of the system and then to predict future states for different scenarios of development. The present state is related to past states by simulation of the historical record, and future states are predicted from projection of the simulation. The simulation employs only reasonable values of the variables; unrealistic values, chosen to force a fit between the record and the simulation, are eschewed. Nor are non-rational formulations of the variables permitted.

The possible future development scenarios for Southern Oahu are too numerous to simulate, but the chief changes likely to occur are as follows:

A. Pearl Harbor Region
   1. Increase in draft
      a. Plantation
      b. BWS
      c. Other
   2. Reduction in draft
      a. Plantation
   3. Reduction in Waiahole Ditch component
   4. Reduction of natural inflow
      a. Increased draft at Wahiawa
   5. Conversion from furrow to drip irrigation
   6. Increase in inflow
      a. Recharge impoundments
      b. Wastewater usage

B. Honolulu
   1. Increase in draft
   2. Reduction in draft
   3. Increase in inflow
      a. Recharge impoundments

The model predicts the time required for the system to adjust to changes in operation, behavior during the period of change, and either the final equilibrium storage state (possible only if input is greater than output) or the time at which the system is destroyed (when output is greater than input).

Model definitions

Description and discussion of actual production, potential production and behavior of the aquifers of Southern Oahu are often confusing because definitions of the variables have differed among investigators. Common terms such as **draft** have not been standardized so that, for instance, the statistics of draft in one report may significantly differ from those in another, although the same data were used. The failure of consistency most profoundly affects the determination of standard values for sustainable yield, which itself is frequently poorly defined.

For any analytical model, definitions of the parameters have to be clear and consistent. The robust analytical model employed in this report deals with the relationship among head (h), time (t), draft (D), and natural recharge (I); to satisfy the relationship it must have values for initial head (h_o) and initial storage (V_o). Once the relationships are established and proven, values of leakage (L), change in storage (Δ V) and sustainable yield (SY) may be computed.

A variable whose definition has been problematic is draft. Some consider it as total outflow from the groundwater system, others consider it as total pumpage. In the Pearl Harbor region its definition is further confused because a portion of it recirculates to the aquifer during irrigation, and thus one can refer to either total or net draft.

Imprecision can be avoided by defining the terms used in hydrological models of Southern Oahu as follows:

Total draft — the total quantity of water forcibly pumped from within the aquifer that would not have naturally discharged to the ground surface from the aquifer in the vicinity of the pump.

This definition excludes any groundwater that is freely discharging, such as springs, free flow wells in the spring areas and spring-tunnel flows, even though these waters may pass through a pump in being raised to where they are consumed. Nor does it include stream flows and Waiahole Ditch flow.

Net draft — total draft less that portion of it that infiltrates back to the aquifer during irrigation.

In the analytical model that net draft is employed as the draft term. For computational convenience changes in inputs are incorporated in the net draft term.

Head — level of the water table or potentiometric surface (for artesian conditions) above mean sea level. Storage head (h_v) refers to the theoretical head that would express the true storage state of the lens in its vertical dimension. Operational head (h_p) is the water level that is actually measured; it reflects local and regional transient drawdown caused by pumping.

Because measured head has been uncritically accepted as true storage head, hydraulic analyses have been all but impossible to interpret. As long as pumps are running measured head is transient; when all pumps are shutdown it recovers toward storage head.

Time — a fixed interval, usually taken as one year for convenience.

Infiltration — natural recharge from rainfall and subsurface inflows. Employed as a constant in the model.

Leakage — sum of the outflows that do not require forcible pumping from within the aquifer. Included are springs, seepages into and at the edge of the caprock, free flow watercress wells, and spring-tunnels at Hawaiian Electric Co. Waiau plant and plantation tunnels in Waikele Valley. Leakage is a function of head whereas draft is manipulated.

Storage — the volume of fresh water resident at any moment in the aquifer. Initial storage is the volume that prevailed at initial head before the first well was drilled. Change in storage is the volume removed from or added to storage.

Sustainable yield — the water supply that may normally be withdrawn from a source at the maximum rate which will not unduly impair source utility.

The robust analytical model

Hydrologic budgeting and input-output mass balances have been the common ways of attempting quantification, but physical as well as mathematical hydraulic flow models also have been widely employed. The "mechanical testing program" performed by the Board of Water Supply forty-five years ago (Biennial Report of the Honolulu Water Supply, 1933-1934 and 1935-1936) was the first attempt at applying a mathematical model, and its failure led to the efforts of C. K. Wentworth to establish the physics of groundwater flow in basal aquifers. Wentworth verified the applicability of Darcy's Law in Hawaiian hydrology (Wentworth 1946) and created the concept of bottom storage in an effort to reconcile the apparent lack of correlation between the large volumes of water extracted from the Honolulu aquifers and prevailing heads (Wentworth 1942). He also proposed a "rinsing hypothesis" to explain the formation of the transition zone (Wentworth 1947). The essential correctness of this hypothesis was much later verified in the derivation of the convection-dispersion equations. More recently TEMPO (1974) attempted unsuccessfully to produce a finite difference model combining the flow and transport equations, and both finite element and other finite difference models are being contemplated by the Water Resources Research Center of the University of Hawaii and the U.S. Geological Survey. The Water Resources Research Center pioneered the use of physical models, extending their applicability to the limits of practicality.

Finite difference and finite element representations of aquifer dynamics appear ready to dominate the field of quantitative groundwater studies. These are sophisticated models aimed at providing the distribution of head and salt (chlorides) throughout an aquifer under variable natural and artificial stresses, such as recharge and pumping. Complete and even practical solutions employing these techniques are subject to formidable obstacles when an aquifer system is complex, such as are the basal aquifers of Hawaii.

In the meantime the application of classical mathematical analysis has not been exhausted by any means in describing and predicting the behavior of Ghyben-Herzberg systems in Hawaii. The type of data reported by water users and collected by the state and the U.S. Geological Survey for more than 50 years lends itself to use in a robust analytical model that combines the continuity and fundamental Darcy equations but does not take into account transport phenomena (convection and dispersion). The robust approach is suited to thick basal lenses like those of Oahu. A robust model assumes a sharp interface, a realistic assumption for a lens having a head of five feet or more and a large groundwater flux.

In the robust analytical model, $h = f(t, D, I, h_o, V_o)$ in which h is head, t is time, D is net draft, I is natural recharge, h_o is initial head, and V_o is initial volume of water in storage. Head and time are continuous variables, and draft can be varied by discretizing time into fixed intervals. Natural recharge, initial volume and initial head are fixed constants. The derivation of the model is given in Appendix III.

The model is structured to solve for head at any time in the history of development for sequences of average draft in fixed intervals, usually one year. For non-equilibrium states different equations are needed for the following conditions (I = natural recharge, D = net draft):

$I > D$

$0 < I < D$

$I = D$

The appropriate equations are given in Appendix III. Only the condition, I>D, eventually settles at a steady state. The prevalence of the other two conditions would eventually destroy the groundwater system.

In all of the equations of the model head is equivalent to storage head. Although theoretically it would be possible to derive a mathematical relationship between storage and operating heads, it is, in reality, all but impossible to do so in Southern Oahu where the boundaries and aquifer parameters are incompletely known and the pattern of draft is complicated. Nevertheless, an attempt was made to determine change in head due to change in draft both theoretically (image method) and empirically (analysis of records for periods when virtually all draft ceased and the aquifer recovered). These approaches are discussed in the section on "Storage and Operating Heads in the Pearl Harbor Aquifer."

Application of the model to Southern Oahu

The most comprehensive compilation of draft data in Hawaii for the period since about 1900 has been provided by the sugar plantations. Heads have been reported by plantations as well as by several government agencies, initially the U.S. Geological Survey, then the Honolulu Board of Water Supply, and more recently the State. The data set for the Pearl Harbor region is the most complete in the State and has been compiled in a form adaptable to easy use in the model equations.

Simulation 1916-1978; 1938-1978

Two simulation cases in which physical boundaries of the aquifer are different are illustrated to show the correspondence between actual and computed trends for periods of known draft. In one of the simulations all of the Koolau basalt aquifer from Manoa Valley to the Koolau-Waianae unconformity west of Waikele Stream is treated as a single aquifer, while in the other the combined Koolau and Waianae aquifers of the Pearl Harbor region are taken as a unit.

Well 244 in Waipahu (replaced in recent years by Well 241) is used as the primary index well, and Well 276 on the Ewa Plain as a supplementary index. The computed and measured maximum and minimum heads for each year are plotted. The program is written for average annual net draft (D), which is the total draft less the component of irrigation that returns to the basal aquifer. The minimum measured head reflects maximum drawdown, while the maximum measured head, although an operating head (h_p), more nearly corresponds to the computed storage head (h_v).

The program requires values for constant recharge (I), initial head (h_o), head at the start of the simulation (h_i), a constant time interval ($t_{i+1} - t_i$), average annual net draft (D), and the initial volume of water in the aquifer (V_o). Heads are measured in feet, time in days, recharge in ft³/day, draft in each interval in ft³/day, and storage volume in ft³.

In case 1 (Pearl Harbor combined Koolau and Waianae aquifers, Red Hill to the Waianae crest) the simulation starts in 1916 when heads had recovered almost to their initial levels because of an extended period of almost zero draft induced by ample rainfall over the plantations. The conditions for the simulation (Fig. 6, Table 16) are as follows:

$I = 29.4 \times 10^6$ ft³/d (220 mgd)
$t_{i+1} - t_i = 365$ days
h_o (Well 244) = 33.5 ft
h_o (Well 276) = 20.0 ft
h_i (Well 244) = 31.0 ft (1916)
h_i (Well 276) = 17.0 ft (1916)
$V_o = 350 \times 10^9$ ft³

TABLE 16

Simulation 1916-1978

Pearl Harbor Region (Red Hill to Waianae Crest)

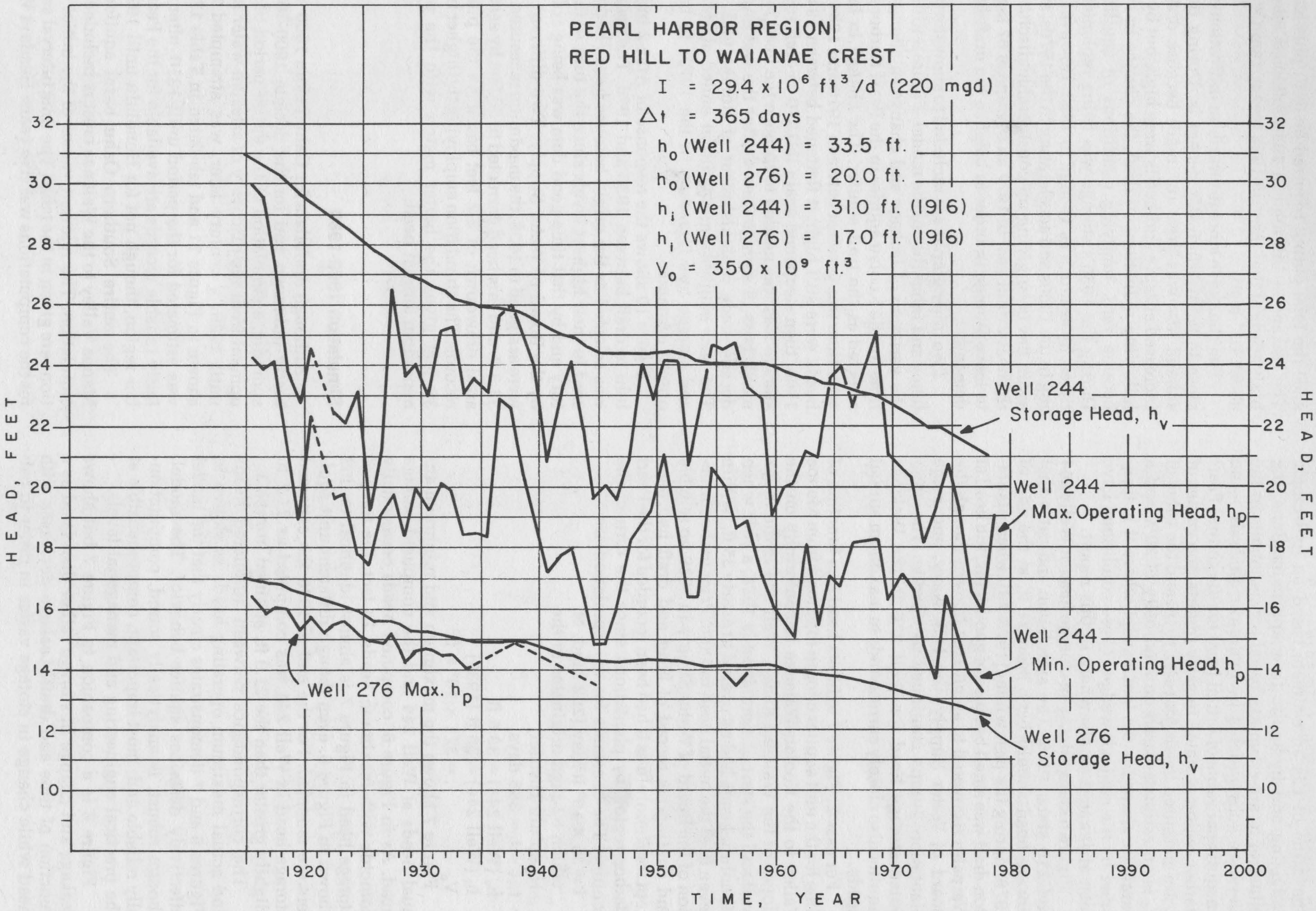
Conditions (draft in mgd; head in ft.)

D = average net draft
I = 220 mgd
Δt = 365 days (1 yr.)
h_o (Well 244) = 33.5 ft.
h_i (Well 244) = 31 ft. (1916)
$V_o = 350 - 10^9$ ft³

h_o (Well 276) = 20.0 ft.
h_i (Well 276) = 17 ft. (1916)

Year	Net D	h	Year	Net D	h	Year	Net D	h	Year	h	Year	h	Year	h
1916	100	30.7	1938	111	25.7	1961	135	23.7	1916	16.9	1938	14.9	1961	13.9
1917	87	30.5	1939	117	25.6	1962	136	23.6	1917	16.8	1939	14.8	1962	13.9
1918	82	30.3	1940	126	25.4	1963	113	23.6	1918	16.8	1940	14.7	1963	13.9
1919	143	29.8	1941	150	25.2	1964	146	23.4	1919	16.6	1941	14.6	1964	13.8
1920	103	29.5	1942	136	25.0	1965	120	23.4	1920	16.5	1942	14.5	1965	13.8
1921	109	29.3	1943	133	24.8	1966	151	23.2	1921	16.4	1943	14.4	1966	13.7
1922	125	28.9	1944	148	24.6	1967	133	23.1	1922	16.2	1944	14.3	1967	13.7
1923	117	28.7	1945	141	24.4	1968	129	23.1	1923	16.1	1945	14.2	1968	13.6
1924	124	28.4	1946	101	24.4	1969	151	22.9	1924	16.0	1946	14.2	1969	13.5
1925	138	28.0	1947	106	24.4	1970	176	22.6	1925	15.8	1947	14.2	1970	13.4
1926	143	27.7	1948	96	24.4	1971	171	22.4	1926	15.6	1948	14.3	1971	13.2
1927	104	27.5	1949	111	24.4	1972	168	22.2	1927	15.6	1949	14.3	1972	13.1
1928	134	27.2	1950	100	24.4	1973	180	21.9	1928	15.5	1950	14.3	1973	13.0
1929	136	26.9	1951	92	24.5	1974	134	21.9	1929	15.3	1951	14.3	1974	13.0
1930	107	26.8	1952	135	24.3	1975	161	21.7	1930	15.3	1952	14.2	1975	12.9
1931	143	26.5	1953	148	24.1	1976	177	21.5	1931	15.1	1953	14.1	1976	12.8
1932	117	26.4	1954	118	24.1	1977	196	21.2	1932	15.1	1954	14.1	1977	12.6
1933	136	26.1	1955	108	24.1	1978	174	21.0	1933	15.0	1955	14.1	1978	12.5
1934	115	26.0	1956	112	24.1				1934	14.9	1956	14.1		
1935	108	25.9	1957	131	24.0				1935	14.9	1957	14.0		
1936	105	25.8	1958	87	24.1				1936	14.9	1958	14.1		
1937	96	25.8	1959	122	24.0				1937	14.9	1959	14.1		
			1960	137	23.9						1960	14.0		

FIG. 6



PEARL HARBOR REGION
RED HILL TO WAIANAE CREST
$I = 29.4 \times 10^6 \; ft^3/d$ (220 mgd)
Δt = 365 days
h_o (Well 244) = 33.5 ft.
h_o (Well 276) = 20.0 ft.
h_i (Well 244) = 31.0 ft. (1916)
h_i (Well 276) = 17.0 ft. (1916)
$V_o = 350 \times 10^9 \; ft.^3$
Well 244 Storage Head, h_v
Well 244 Max. Operating Head, h_p
Well 244 Min. Operating Head, h_p
Well 276 Storage Head, h_v
Well 276 Max. h_p
HEAD, FEET
TIME, YEAR
32
30
28
26
24
22
20
18
16
14
12
10
1920
1930
1940
1950
1960
1970
1980
1990
2000

The value of I is an estimate based on hydrologic budgeting and hydraulic flow approximations; the values of h_o are estimates based on comments in the literature substantiated by hydraulic analysis — no unequivocal value of initial head for the central Pearl Harbor region is stated in early reports; the values of h_i are measured heads for early 1916; and the value of V_o is an estimate based on a porosity of 10% and a parabolic shape for the lens except where it is truncated by the caprock wedge. These conditions have been elaborated on elsewhere in this report.

Figure 6 shows decay in storage head at Wells 244 and 276 since 1916. The simulation indicates that storage head should have been 21 ft. at the end of 1978; during the past winter (1978-79), when plantation draft was small because of good rains, the head at Waipahu recovered to about 19.2 ft. even though the Board of Water Supply, the U.S. Navy, and some plantation pumps continued to operate. The computed storage head for Well 276 in the Waianae Aquifer also closely corresponds to maximum annual heads.

For case 2, Figure 7 exhibits head as a function of time for the unit aquifer chosen to extend from Manoa Valley to the Koolau-Waianae unconformity on the slopes of the Waianae Range. Well 244 is the index well and the start of simulation is 1938, a year when Honolulu heads had recovered to nearly 35 ft., within seven ft. of the initial head in 1880. Prior to the formation of the Board of Water Supply in 1929 no reliable and continuous record of head and draft had been kept for Honolulu as it had been recorded for the Pearl Harbor region by plantations since the turn of the century. The constants for the simulation are

$I = 35 \times 10^6$ ft^3/day (262 mgd, of which 62 mgd originates in the Honolulu District)

$t_{i+1} - t_i = 365$ days

h_o (Well 244) = 33.5 ft

h_i (Well 244) = 26 ft (1938)

$V_o = 371 \times 10^9 \times$ ft^3

Figure 7 shows the maximum and minimum annual heads at Well 244 and the computed storage head. As in Figure 6, computed heads occasionally coincide with maximum heads. In fact, the trace of storage head in Figure 7 is almost identical to that shown in Figure 6, even though different unit aquifers are modeled. For the end of 1978 the expected storage head at Well 244 was computed as 21.2 ft. slightly greater than the 21.0 ft. attained in case 1.

The correspondence between simulated heads and actual maximum operating heads as shown in Figures 6 and 7 demonstrates clearly that the model effectively describes aquifer behavior. The model, though robust, is analytically sound, computationally reliable and, most important, comprehensible at the practical engineering and managerial levels.

Figure 8 is a companion to Figure 7 but shows leakage and change in storage rather than head as a function of time and draft. Leakage decreases with head while change in storage varies in order to balance the total output required by draft plus leakage. The leakage of 153 mgd in 1938 reflected relatively high heads of that time; in 1978 leakage amounted to about 107 mgd.

The change in storage term is an indication of how close to equilibrium the system is. During the war storage loss was great (67 mgd) because draft had increased sharply and heads were high, but by 1950, following reductions in draft, the change in storage became zero, implying conditions of equilibrium. During the 1950's storage was being replenished at times, by as much as 19 mgd in 1958. However, starting in the 1960s and accelerating in the 1970s, storage again has been giving up considerable discharge. In the heavy draft year of 1977 an average of 67 mgd had to leave storage in order to balance the draft-leakage demands.

Two other graphs are included to show the consistency and reliability of the model. Figure 9 illustrates the variation of leakage and change in storage with time for the Koolau aquifer in the Pearl Harbor region selected as the unit aquifer. The change in leakage with time was greatest between 1916 and 1934 when heads were still high; it flattened between 1942 and 1966, then steepened again in 1970 as draft rose and storage loss was needed to balance the head requirement. Loss of storage ceased after the war and for a decade some replenishment of storage took place. The near equilibrium condition ended about 1960, and storage loss increased in the 1970s to balance output demands.

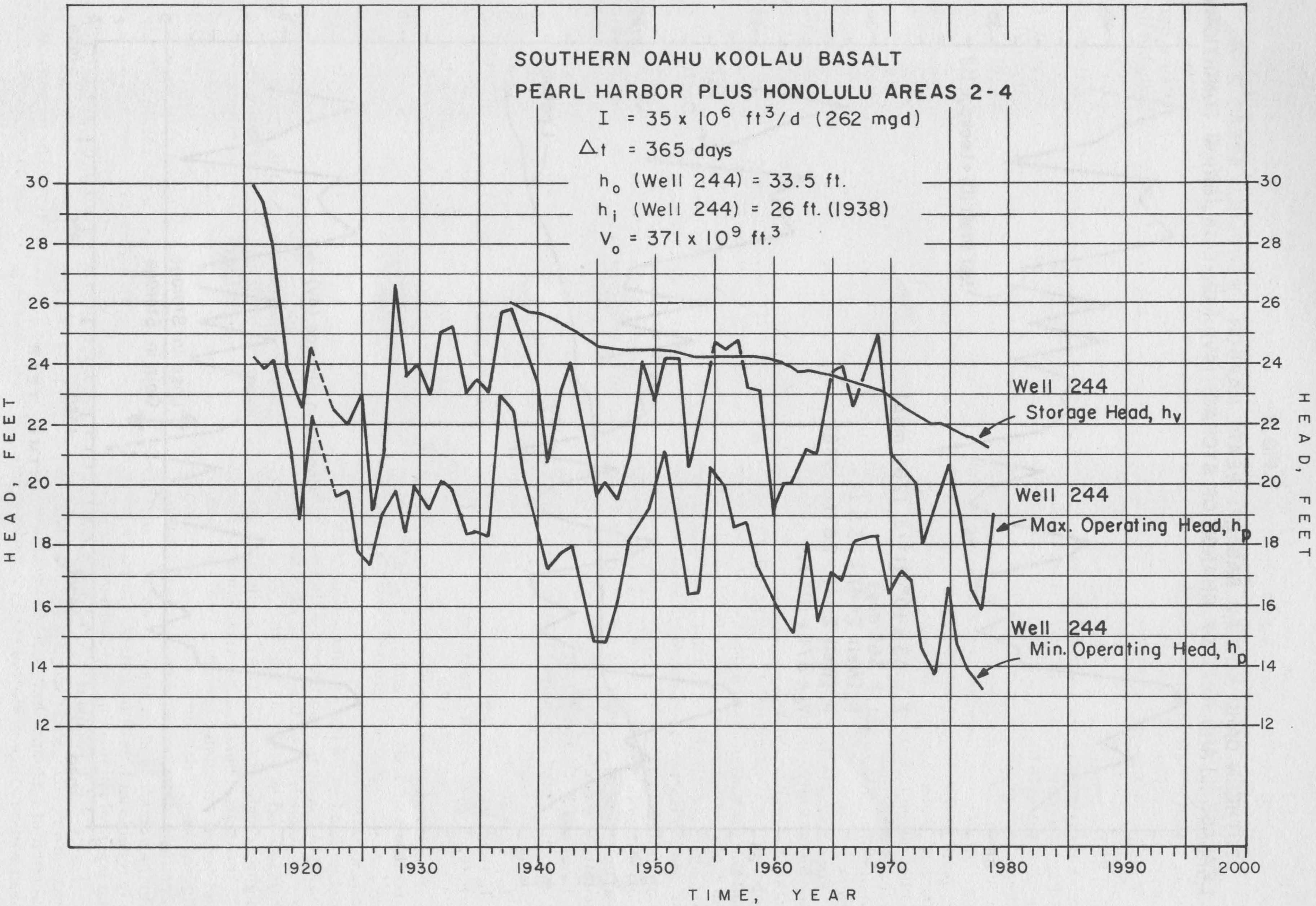
Figure 10 shows the response of Area 2 in Honolulu to draft between 1938 and 1978. The year 1938 was selected as the origin because heads had recovered to their highest levels since the turn of the century and by that time good data was being collected by the Board of Water Supply. Two different heads were assigned to 1938, the maximum measured one of 33.3 ft. and this head corrected to 36 feet by assuming areal drawdown of 2.7 feet based on the pumping records. The simulation employing the higher head of 36 feet gives the better match with the plot of maximum annual head.

Simulation 1880-1980

Although draft data for Oahu before 1900 are virtually unknown and for the decade 1900-1910 are sporadic, a simulation of the entire period of development from the discovery of artesian water in 1879 until 1980, a century later, was attempted and is shown in Figure 11 and tabulated in Table 17. Draft was estimated for the period until 1910, after which fairly reliable records are available for the Pearl Harbor region, though not for Honolulu until 1928.

The entire Southern Oahu basal aquifer from Manoa Valley to the Waianae crest is included in the simulation. The relevant values of the initial conditions are given in the table. The time interval selected for the computations was five years. Heads at Well 83

FIG. 7



SOUTHERN OAHU KOOLAU BASALT
PEARL HARBOR PLUS HONOLULU AREAS 2-4
I = 35 x 10^6 ft^3/d (262 mgd)
Δt = 365 days
h_o (Well 244) = 33.5 ft.
h_i (Well 244) = 26 ft. (1938)
V_o = 371 x 10^9 ft.^3
Well 244
Storage Head, h_v
Well 244
Max. Operating Head, h_p
Well 244
Min. Operating Head, h_p
HEAD, FEET
HEAD, FEET
TIME, YEAR
30
28
26
24
22
20
18
16
14
12
1920
1930
1940
1950
1960
1970
1980
1990
2000

FIG. 8

SOUTHERN OAHU KOOLAU BASALT: PEARL HARBOR + HONOLULU AREAS 2-4
LEAKAGE (L) AND RATE OF CHANGE OF STORAGE (ΔV): NON-EQUILIBRIUM CONDITIONS

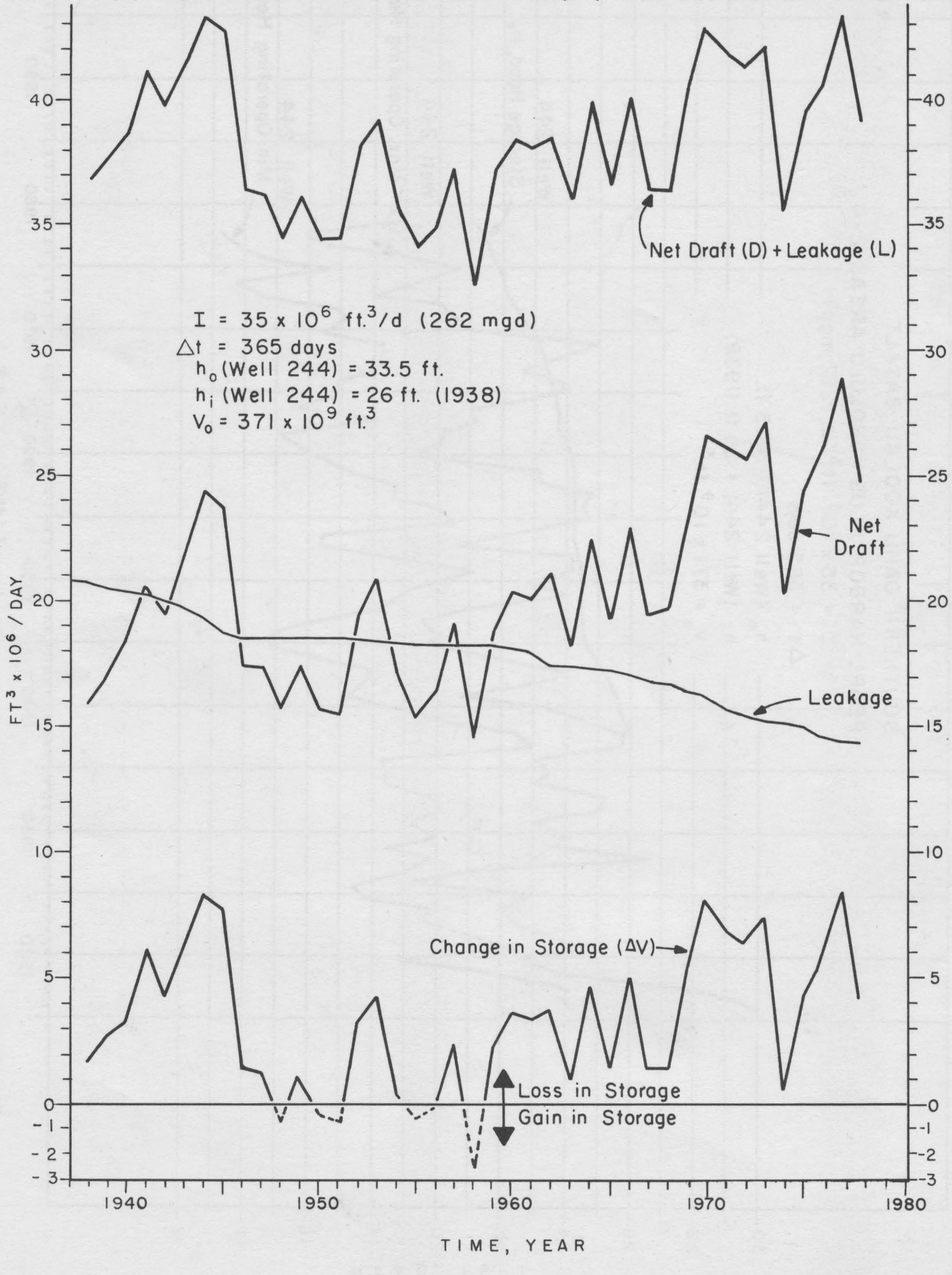
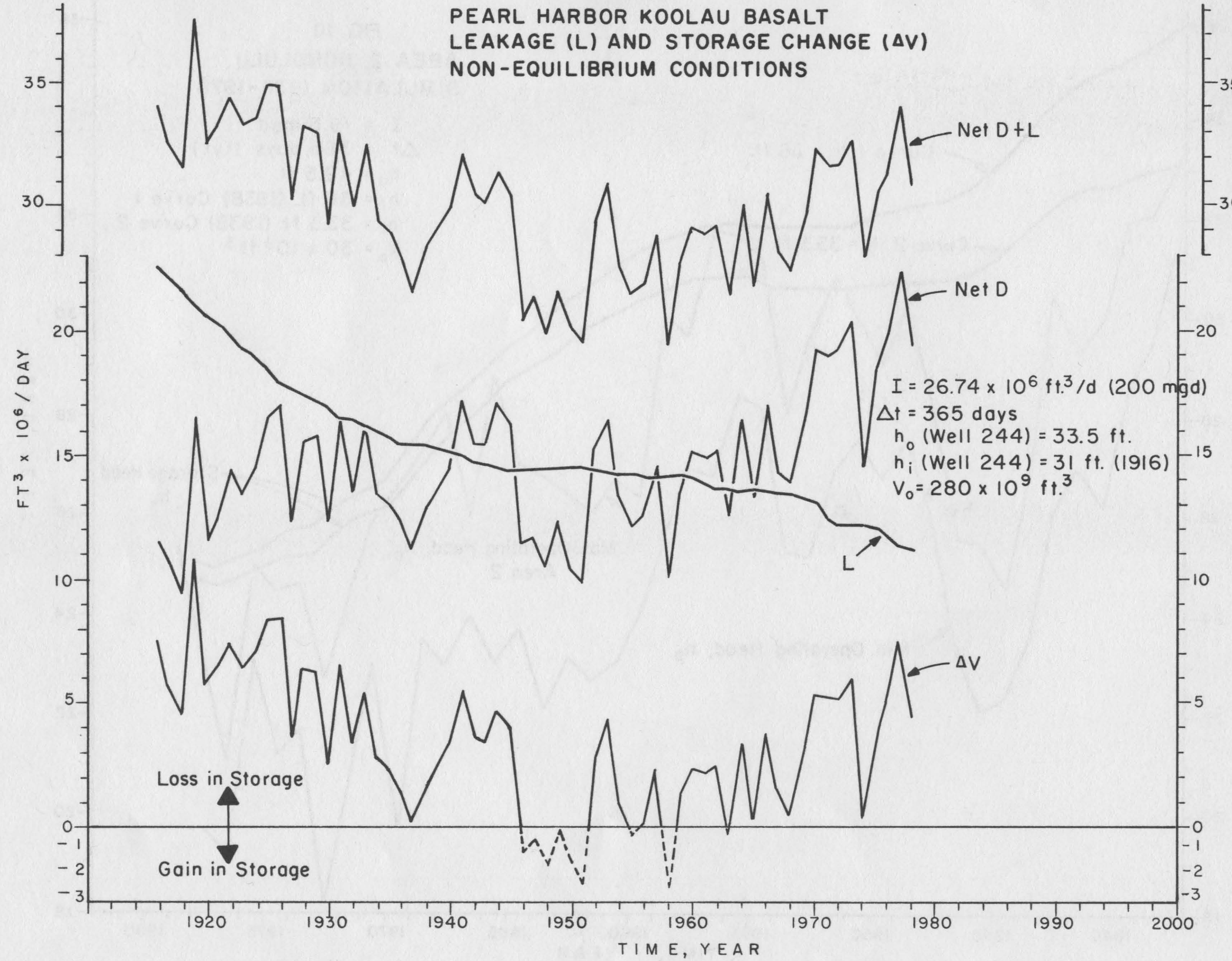


FIG. 9

PEARL HARBOR KOOLAU BASALT
LEAKAGE (L) AND STORAGE CHANGE (ΔV)
NON-EQUILIBRIUM CONDITIONS



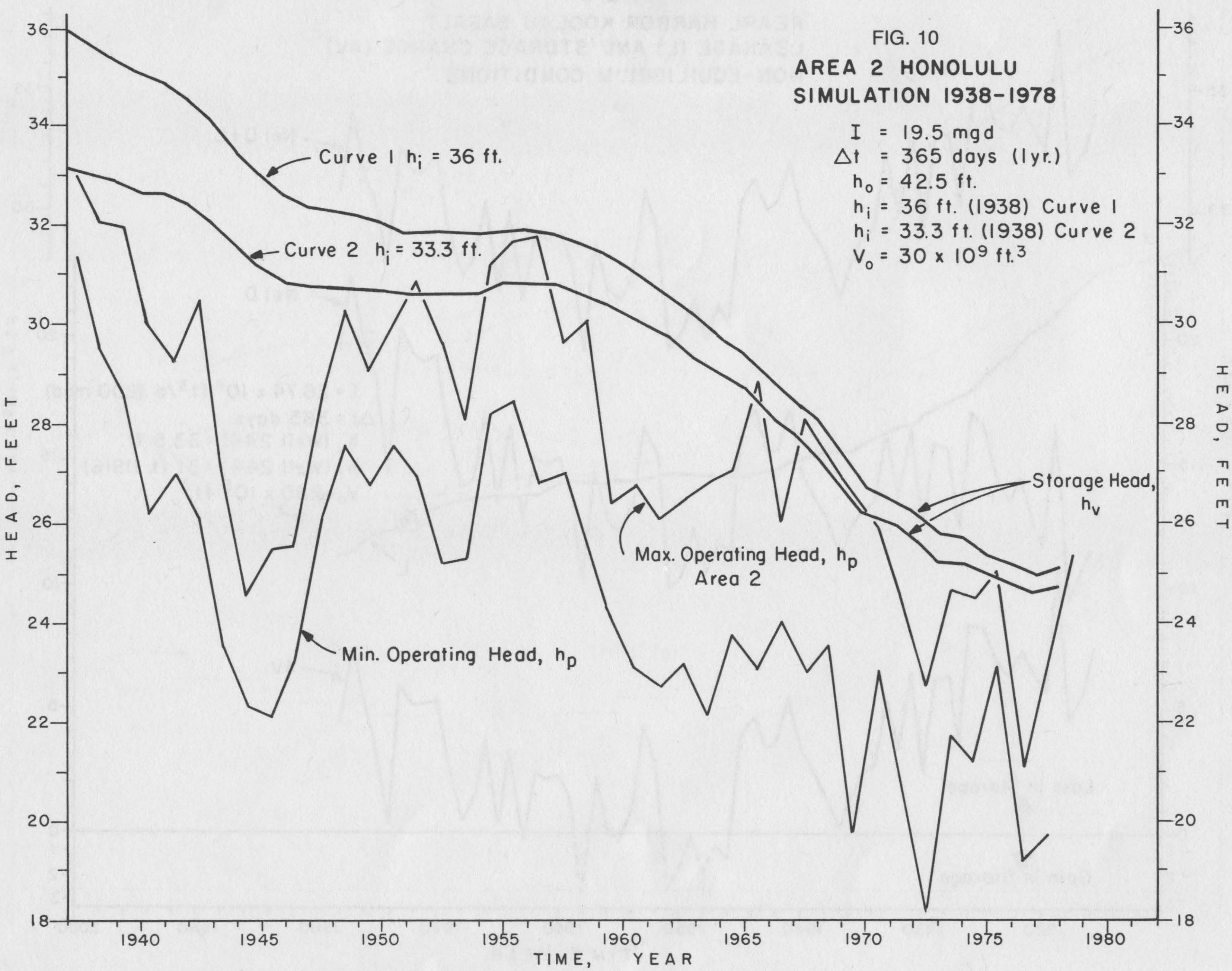


FIG. 10
AREA 2 HONOLULU
SIMULATION 1938-1978
I = 19.5 mgd
Δt = 365 days (1yr.)
h_o = 42.5 ft.
h_i = 36 ft. (1938) Curve 1
h_i = 33.3 ft. (1938) Curve 2
V_o = 30 x 10^9 ft.^3
Curve 1 h_i = 36 ft.
Curve 2 h_i = 33.3 ft.
Storage Head, h_v
Max. Operating Head, h_p
Area 2
Min. Operating Head, h_p
HEAD, FEET
HEAD, FEET
TIME, YEAR
1940
1945
1950
1955
1960
1965
1970
1975
1980
18
20
22
24
26
28
30
32
34
36

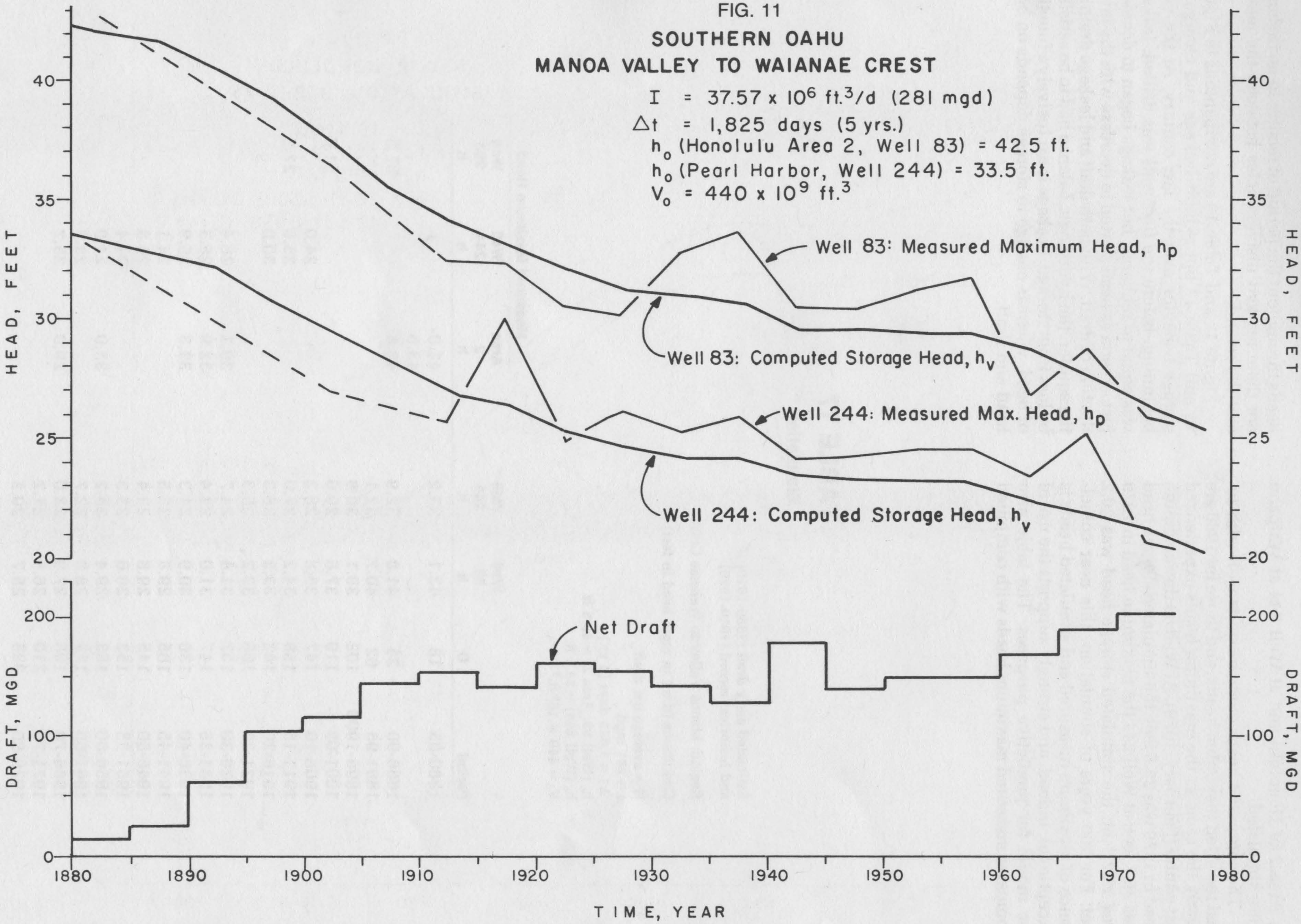


FIG. 11
SOUTHERN OAHU
MANOA VALLEY TO WAIANAE CREST

in Area 2 of Honolulu and at Well 244 at Waipahu were simulated.

The computed heads compare quite well with actual recorded maximum heads, and those for 1980 are nearly the same as the maximum heads experienced last winter (February, 1979). At Well 83 the maximum head in 1979 was 25.5 feet, the simulated storage head was 25.7 feet; at Well 244 the maximum head in 1979 was 19.2 feet, the simulated storage head was 20.2 feet. For 100 years of simulation, this near coincidence of maximum measured and simulated heads is exceptional, indeed, and strongly supports the use of the model for predictive purposes. The table also compares measured maximum heads with computed heads throughout the period of record; in no instance are there marked discrepancies between the model and the record.

Figure 12 and Table 18 are companions to Figure 11 and Table 17 but exhibit leakage and change in storage behavior over the last century. At the very beginning, before the first well was drilled, leakage was equal to recharge, but leakage began to decrease just as soon as head started to decrease with the introduction of draft. With both draft and leakage draining the aquifer, the hydrologic balance had to be satisfied by loss from storage. Leakage is exclusively a function of head, whereas change in storage depends on both head and draft.

TABLE 17

Simulation

Estimated early draft (1880-1910)
and historical record (1910-1979)

Region: Manoa Valley to Waianae Crest

Conditions (draft in mgd; head in feet)

D = average net draft
I = 281 mgd
Δt = 1,825 days (5 yr.)
h_o (Well 83, Area 2) = 42.5 ft.
h_o (Well 244) = 33.5 ft.
$V_o = 440 \times 10^9$ ft.^{3}

				Measured Maximum Head		
Period	D	Well 83 h	Well 244 h	Area 2 h	Well 244 h	Well 268 h
1880-85	18	42.1	33.2	42.0-43.5		
1886-90	24	41.8	32.9	42.8		31.5
1891-95	62	40.7	32.1			
1896-1900	103	39.1	30.8			
1901-05	119	37.5	29.6			31.6
1906-10	147	35.8	28.2		24.0	
1911-15	158	34.2	27.0		25.8	27.9
1916-20	142	33.3	26.3		30.0	
1921-25	165	32.2	25.3			
1926-30	157	31.4	24.7	30.1	26.4	
1931-35	147	31.0	24.4	32.9	25.3	
1936-40	136	30.9	24.3	33.3	25.9	
1941-45	185	29.8	23.5		24.1	
1946-50	145	29.8	23.4		24.3	
1951-55	153	29.6	23.3		24.4	
1956-60	153	29.4	23.2	33.0	24.8	
1961-65	173	28.9	22.7		23.7	
1966-70	195	27.9	22.0	29.0	25.2	
1971-75	210	26.8	21.2			
1976-80	225	25.7	20.2			

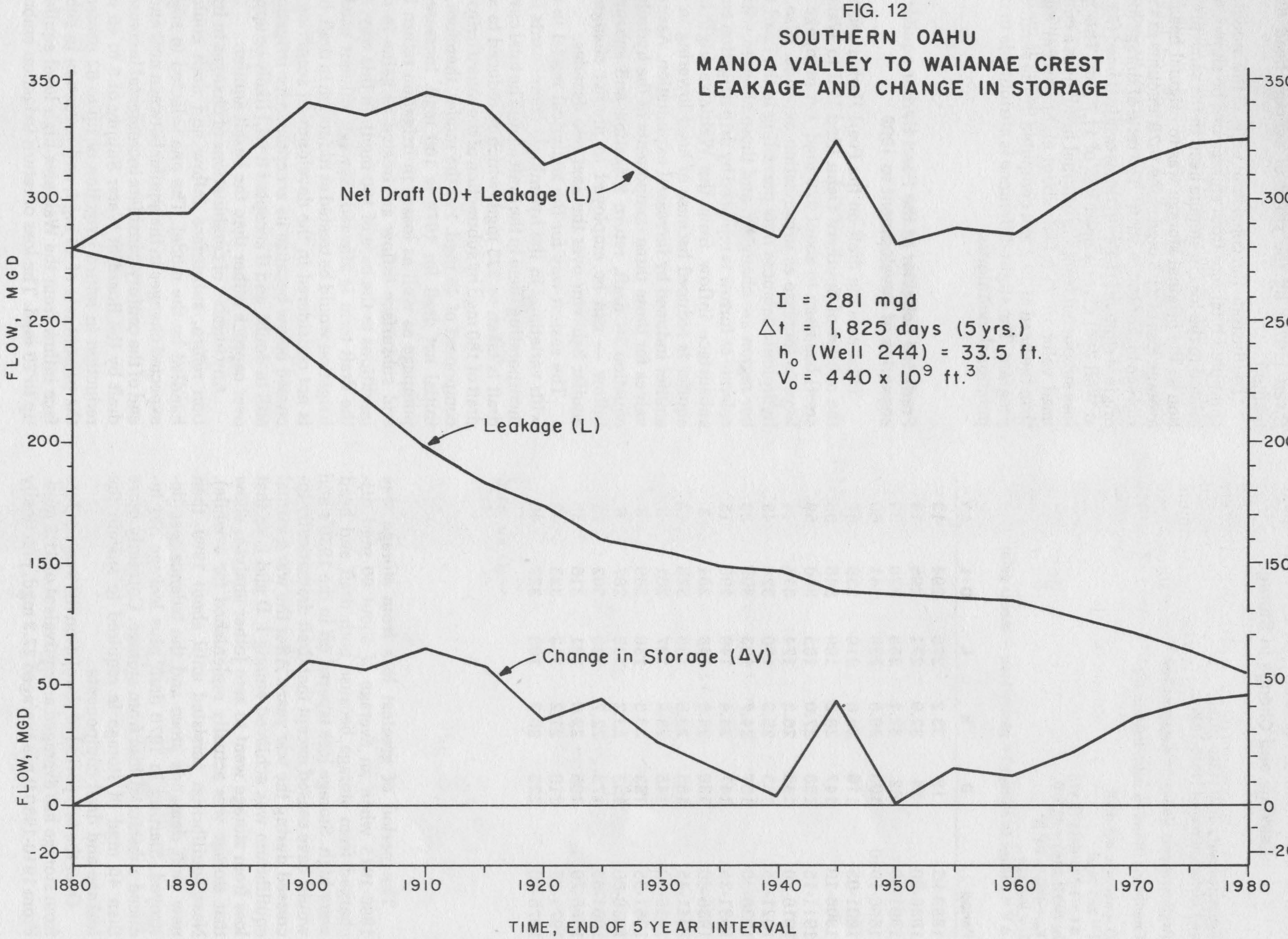


FIG. 12
SOUTHERN OAHU
MANOA VALLEY TO WAIANAE CREST
LEAKAGE AND CHANGE IN STORAGE

TABLE 18

Leakage and Change in Storage

Estimated early draft (1880-1910)
and historical record (1910-1979)

Region: Manoa Valley to Waianae Crest

Conditions (flows in mgd; head in ft.)

D = average net draft
I = 281 mgd
Δt = 1,825 days (5 yr.)
h_o (Well 244) = 33.5 ft.
$V_o = 440 \times 10^9$ ft^3
L = leakage
ΔV = change in storage (+ means loss; − means gain)

Period	D	h	L	D+L	ΔV
1880-95	18	33.2	276	294	13
1886-90	24	32.9	271	295	14
1891-95	62	32.1	258	320	39
1896-1900	103	30.8	238	341	60
1901-05	119	29.6	219	338	57
1906-10	147	28.2	199	346	65
1911-15	158	27.0	183	340	59
1916-20	142	26.3	174	315	34
1921-25	165	25.3	160	324	43
1926-30	157	24.7	153	309	28
1931-35	147	24.4	149	296	15
1936-40	136	24.3	148	284	3
1941-45	185	23.5	138	323	42
1946-50	145	23.4	137	281	0
1951-55	153	23.3	136	289	8
1956-60	153	23.2	135	287	6
1961-65	173	22.7	129	302	21
1966-70	195	22.0	121	316	35
1971-75	210	21.2	113	323	42
1976-80	225	20.2	103	327	46
					av. 31.5

The period of greatest loss from storage was 1896-1915 when an average of about 60 mgd discharged from storage because both draft and head were high. Storage loss tapered off in the 1930's and would have ceased except that draft dramatically increased during the war years. After the war a virtual equilibrium was achieved among I, D and L so that loss from storage went to zero (other analyses show that storage was actually replenished for a while). Near equilibrium persisted until about 1960, then new draft demands arose and the balance was destroyed. Starting in 1970 draft plus leakage has induced substantial loss from storage. Currently more than 45 mgd of storage is required to sustain the leakage and draft components.

Over the full period of simulation the average loss from storage has averaged an equivalent of 31.5 mgd. From 1910-1980 it has averaged 27.3 mgd, practically the same as the 25 mgd computed by Soroos and Ewart (1979) by the slope of average head decay method.

To illustrate the consistent logic of the model, the computed total loss from storage can be shown to be related to the loss in storage head in the same proportion as the original storage was to original head. An average loss of 31.5 mgd since 1879 amounts to a total storage loss of 154×10^9 ft^3. The ratio of storage loss to original storage (154/440) times original head (33.5 ft. at Well 244) gives a head loss of 11.7 feet. This head loss subtracted from the original head gives a current head value of 21.8 feet, about eight percent higher than the head of 20.2 feet computed by the simulation program. The slight difference is attributable to computational techniques.

Predicted behavior of the Pearl Harbor aquifer for scenarios of development to 1999

Total average draft on the Pearl Harbor sector of the aquifer of Southern Oahu (Red Hill to Waianae crest) has risen to about 220 mgd, yet pumpage will have to increase as urbanization expands. Also, the hydrologic balances now prevailing in the Pearl Harbor region are changing, and these changes will accelerate as furrow is replaced by drip irrigation and as subsurface inflow from the Wahiawa high level aquifer is reduced because of head lowering in that aquifer induced by increased exploitation. Assumed values for these three components of the hydrologic equation — draft, return irrigation and subsurface inflow — can be employed to predict changes in aquifer behavior over the next two decades.

The easiest way for the analytical model to deal with variations in the hydrologic components is by incorporating them in the draft term. The total current draft is taken as 225 mgd, which is reduced to a net draft of 180 mgd by subtraction of the return irrigation component of 45 mgd. For the model, therefore, the initial net draft for 1979 is 180 mgd. Increases in pumpage as well as losses in irrigation return flow and subsurface inflow are treated as gains in draft (additions to the base of 180 mgd). In this way, only the draft term is affected. Sewage effluent used for irrigation would be treated as reduction in draft, but it is not considered in the development scenarios discussed below because its acceptance for irrigation is still in doubt, and if accepted it will likely be applied over caprock rather than the basalt aquifer.

Any number of combinations of changes in irrigation return, subsurface inflow and draft could be handled by the model. The one selected to suggest expected changes in the aquifer between now and the end of the century considers incremental increases in draft by the Board of Water Supply of 5 to 45 mgd, reduction in return irrigation of up to 62 percent of the present rate of 45 mgd, and reduction in subsurface inflow from the Wahiawa high level aquifer by up to 20 mgd. The loss of return irrigation embraces possible diversions from agricultural use in Southern Oahu of the Waiahole Ditch system.

The area for which the hydrologic variables are assigned extends from Red Hill to the Waianae crest and includes both the Koolau and Waianae aquifer of the Pearl Harbor region. The fixed parameters are:

V_o (initial volume)	$= 350 \times 10^9$ ft^3
I (natural recharge)	= 220 mgd
h_o (initial head Well 244)	= 33.5 feet
h_i (1979 head Well 244)	= 21.0 feet
D (net draft 1979)	= 180 mgd

Net pumped draft is assumed to increase over the base of 180 mgd in the following increments:

Year	Increment (mgd)	Accumulated Increments (mgd)
1980	5	5
1982	5	10
1984	5	15
1987	5	20
1989	5	25
1991	5	30
1994	5	35
1996	5	40
1998	5	45

Irrigation return flow is reduced to 17 mgd by 1996, and loss of Wahiawa high level inflow amounts to 20 mgd by 1995. Table 4 lists the assumed changes by year and summarizes the predicted storage heads. The table includes calculations of head for a scenario which assumes that 1979 hydrologic conditions will be constant.

Figure 13 is a graph showing the expected change in storage head at Well 244 for the conditions in Table 19. For Case 1, which assumes persistence of the 1979 conditions for the next 20 years, storage head would decay to 17.9 feet in 1999 and ultimately to a steady state head (h_e) of 14.3 feet. Case 2, which takes into account additions to net pumped draft and loss of irrigation return, yields a head of 14.8 feet in 1999, but ultimately the resource would be destroyed because net draft exceeds recharge. A similar fate would result in Case 3, which includes loss of subsurface flow from Wahiawa.

The heads shown in Table 19 and given above would be storage, not measured, heads. They would have to be corrected in the Halawa area by approximately .03 feet per mgd total Pearl Harbor pumpage and in the Ewa area by about .04 feet per mgd total pumpage. Thus for the Halawa area the correction, to be subtracted from the storage head, would be 6.8 feet for Case 1 and 9.0 feet for Cases 2 and 3; for the Ewa area the respective corrections would be 8.1 and 10.8 feet. In other words, in 1999 the average measured head near Halawa would be 11.1 feet for Case 1, 6.7 feet for Case 2 and 5.9 feet for Case 3, while near Ewa the respective measured heads would be 8.9 feet, 4.0 feet and 3.2 feet.

Clearly, neither Case 2 nor Case 3 are acceptable scenarios of exploitation. For Case 1, the extraction and use of groundwater retained exactly as at present, the lens would be preserved. An acceptable scenario might allow somewhat more net draft than the 180 mgd (equivalent to total draft of 225 mgd) assigned for Case 1. However, none of the components affecting the net draft term can be viewed in isolation when making decisions; all of the components need to be considered simultaneously.

Figure 13 and Table 19 illustrate how relatively slowly the lens responds to additions in net draft. This characteristic enables the resource to be effectively manageable over a wide range of conditions. For instance, reasonable over-exploitation for as long as a decade might be permissible provided adjustments were made to allow the system to recover after storage head reached a target low.

Figure14 shows anticipated leakage and storage loss for each case discussed above. To satisfy the net drafts of Cases 2 and 3 water is extracted from storage at increasing rates to the end of the century. Sometime beyond the year 2000 all storage would be consumed, but even before that happened the lens would be salinized as its dimensions contracted. In Case 1 loss of storage would decrease gradually until finally at equilibrium no further loss would occur. At equilibrium the total leakage would amount to 40 mgd, the difference between constant net draft of 180 mgd and natural recharge of 220 mgd.

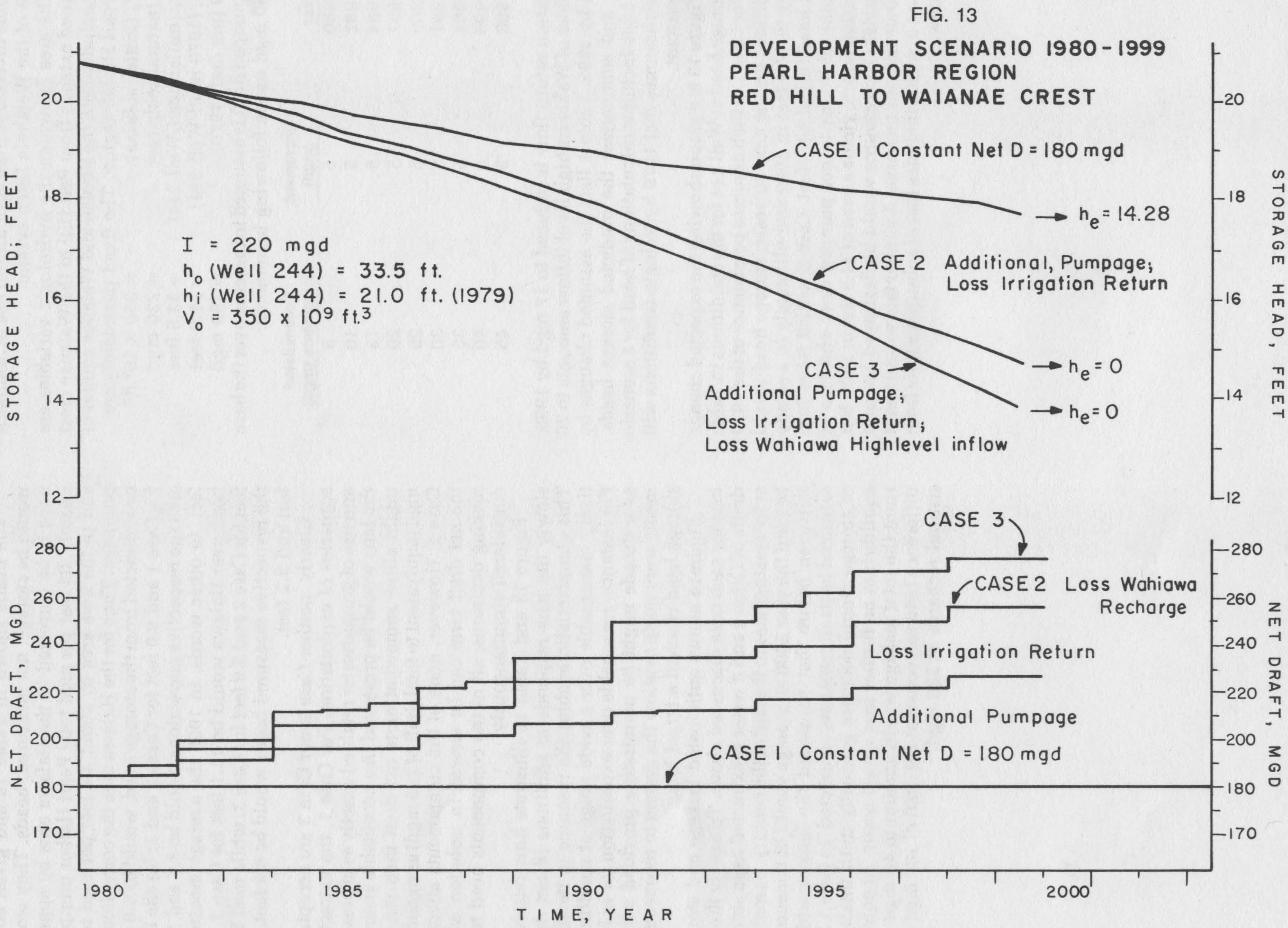


FIG. 13

DEVELOPMENT SCENARIO 1980-1999
PEARL HARBOR REGION
RED HILL TO WAIANAE CREST

TABLE 19

Development Scenario 1980-1999

Assumed increases in net draft caused by increased pumping, loss of irrigation return flows, and loss of Wahiawa high level inflows

Region: Red Hill to Waianae Crest

Conditions (flows in mgd; head in ft.)

D = average net draft
I = 220 mgd Δt = 365 days (1 yr.)
h_o (Well 244) = 33.5 ft.
h_i (Well 244) = 21.0 ft. (1979)
$V_o = 350 \times 10^9$ ft^3

	Case 1		Case 2				Case 3		
Period	Net D	h	Cum. Add. Pump-age ΔD	Loss Irrig. ΔD	Net D	h	Loss Wahiawa ΔD	Net D	h
1980	180	20.8	5	0	185	20.8	0	185	20.8
1981	"	20.6	5	2.3	187	20.5	0	187	20.5
1982	"	20.4	10	4.5	195	20.3	2.5	198	20.3
1983	"	20.2	10	4.5	195	20.0	2.5	198	20.0
1984	"	20.0	15	9.0	204	19.7	5.0	209	19.7
1985	"	19.8	15	9.0	204	19.5	5.0	209	19.4
1986	"	19.7	15	9.0	204	19.2	7.5	212	19.1
1987	"	19.5	20	11.3	211	18.9	7.5	219	18.8
1988	"	19.3	20	11.3	211	18.6	10	221	18.4
1989	"	19.2	25	16.9	222	18.3	10	232	18.1
1990	"	19.0	25	16.9	222	18.0	10	232	17.7
1991	"	18.9	30	22.5	233	17.6	15	248	17.3
1992	"	18.8	30	22.5	233	17.3	15	248	16.9
1993	"	18.6	30	22.5	233	17.0	15	248	16.5
1994	"	18.5	35	22.5	238	16.6	15	253	16.1
1995	"	18.4	35	22.5	238	16.3	20	258	15.7
1996	"	18.3	40	28.1	248	15.9	20	268	15.3
1997	"	18.1	40	28.1	248	15.6	20	268	14.9
1998	"	18.0	45	28.1	253	15.2	20	273	14.4
1999	"	17.9	45	28.1	253	14.8	20	273	14.0
h_e		14.3				0			0

FIG. 14

DEVELOPMENT SCENARIO 1980-1999
PEARL HARBOR REGION
RED HILL TO WAIANAE CREST
LEAKAGE AND LOSS OF STORAGE

I = 220 mgd
Δt = 365 days (1yr.)
V_o = 350×10^9 ft.^{3}

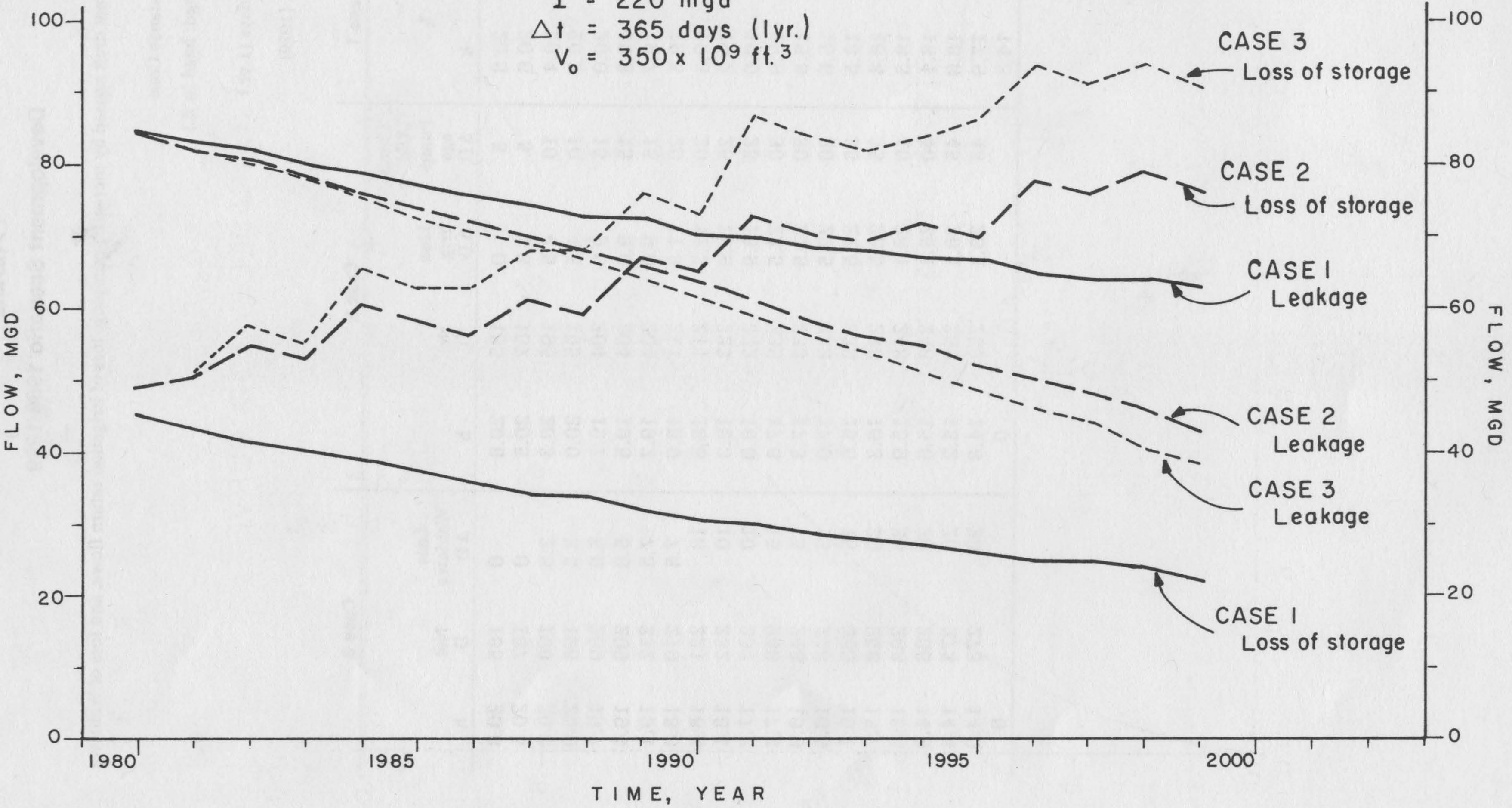
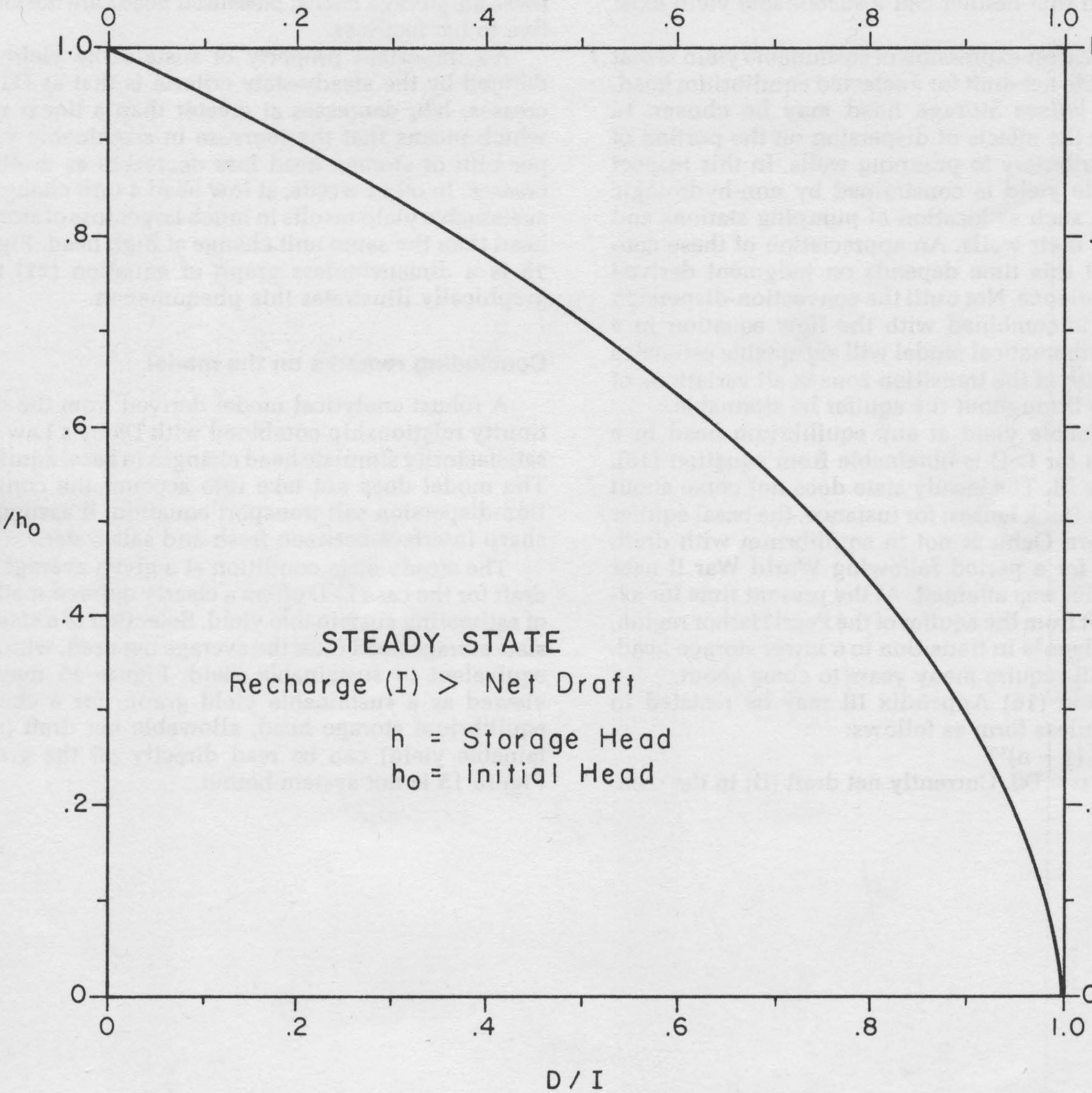


FIG. 15

SUSTAINABLE YIELD CURVE



Sustainable Yields

Sustainable yield is defined as "the water supply that may normally be withdrawn from a source at the maximum rate which will not unduly impair source utility" (State Water Commission, 1979), but in the framework of the foregoing analysis it can be redefined as the allowable net draft at steady state for a selected equilibrium head. Theoretically there are an infinite number of sustainable yields for the condition I>D (natural recharge greater than net draft). Elementary logic states that no sustainable yield is possible for I<D, and on further reflection it must be concluded that neither can a sustainable yield exist for I = D.

The clearest expression of sustainable yield is that of allowable net draft for a selected equilibrium head. In thick lenses storage head may be chosen to minimize the effects of dispersion on the portion of the lens tributary to pumping wells. In this respect sustainable yield is constrained by non-hydrologic variables, such as location of pumping stations and depths of their wells. An appreciation of these constraints at this time depends on judgment derived from experience. Not until the convection-dispersion equation is combined with the flow equation in a useful mathematical model will acceptable estimates of the width of the transition zone at all variations of operation throughout the aquifer be attainable.

Sustainable yield at any equilibrium head in a basal lens for I>D is obtainable from equation (16), Appendix III. The steady state does not come about rapidly in thick lenses; for instance, the basal aquifer of Southern Oahu is not in equilibrium with draft, although for a period following World War II near equilibrium was attained. At the present time for average draft from the aquifer of the Pearl Harbor region, the basal lens is in transition to a lower storage head, which will require many years to come about.

Equation (16) Appendix III may be restated in dimensionless form as follows:

$$h/h_o = (1 - n)^{1/2}$$

in which n = D/I. Currently net draft (D) in the combined Koolau and Waianae aquifers of Pearl Harbor is about 180 mgd. Assuming a constant natural recharge (I) of 220 mgd, at equilibrium h/h_o will be 0.43. If $h_o = 33.5$, the equilibrium head for D = 180 mgd is 14.3 ft. Present transient storage head is about 21 feet. About 19 years will be needed for the storage head to decline to 18 feet for constant net draft of 180 mgd. The equilibrium head is asymptotically approached so that a very long period would pass before the storage head approached 14.5 to 15.0 feet. If net draft increased to and remained at 190 mgd, as it was in 1977, the equilibrium head would be 11.1 feet; a head of 18 feet would be reached in 12.9 years. Note that these are storage heads; measured heads are normally five to ten feet less.

An important property of sustainable yields as defined by the steady-state criteria is that as D/I increases, h/h_o decreases at greater than a linear rate, which means that the increase in sustainable yield per unit of storage head loss decreases as draft increases. In other words, at low head a unit change in sustainable yield results in much larger loss of storage head than the same unit change at high head. Figure 15 is a dimensionless graph of equation (21) that graphically illustrates this phenomenon.

Concluding remarks on the model

A robust analytical model derived from the continuity relationship combined with Darcy's Law can satisfactorily simulate head changes in basal aquifers. The model does not take into account the convection-dispersion salt transport equation; it assumes a sharp interface between fresh and salt water.

The steady-state condition at a given average net draft for the case I>D offers a clearly defined method of estimating sustainable yield. Selection of a steady-state storage head fixes the average net draft, which is equivalent to sustainable yield. Figure 15 may be viewed as a sustainable yield graph; for a chosen equilibrium storage head, allowable net draft (sustainable yield) can be read directly off the graph. Figure 15 is not system-bound.

Appendices

APPENDIX I

Initial Volume of Water in Storage

The volume of water at any instant in an aquifer is called "storage." This volume is not static; it moves slowly down the hydraulic gradient as laminar flow in agreement with Darcy's Law. Only at steady state conditions can it be said that storage is constant. At all other times the volume changes, however minutely, with transient conditions.

The non-steady equation describing change in head with time and draft requires as a constant the initial storage volume of water in the aquifer. Storage in a basal lens depends on aquifer boundaries, lens geometry and effective aquifer porosity. No matter how the volume is computed, a large measure of estimation and judgment is involved because of the complex heterogeneities and configuration of the aquifers.

Basal Aquifer Boundaries of Southern Oahu

Boundaries of the basal aquifer of Southern Oahu, extending from Manoa Valley to the Waianae Range, have been discussed elsewhere in this report (see, for example, the section on Hydrologic Budgets). In summary, the most easterly boundary is Manoa Valley; the Koolau boundary is the end of the marginal dike zone lying about one half mile leeward of the crest; the Wahiawa boundary strikes across the Schofield Plateau presumably along a line straddling the Kaukonahua-Waikakalaua divide and extending to the vicinity of Kunia camp and beyond; the Waianae boundary is approximately coincident with the Waianae Range crest from Puu Kanehoa to Puu Manawalua and parallels Makaiwa Gulch from there southward; and the coastal boundary is the wedge of caprock all along the southern coast of the island. The Manoa, Koolau, Wahiawa and Waianae boundaries are considered to be abrupt, vertical non-basal flow boundaries, while the caprock is treated as a no-flow wedge sloping at an angle of five degrees. In the case of the vertical boundaries, except for Manoa Valley, high level groundwater spills into the basal aquifer. Some groundwater leaks from the aquifer into the Manoa and caprock boundaries.

Hydraulic continuity extends throughout the whole Southern Oahu basal aquifer, but local discontinuities exist that have prompted subdivision of the aquifer into smaller units by various investigators to simplify hydrologic analyses. In particular, the Honolulu District has been divided into five "isopiestic" areas (a misnomer), the the westernmost three of which are closely connected hydraulically (Honolulu Board of Water Supply Areas 2, 3 and 4) to each other and to the Pearl Harbor region in the Halawa area. Area 1, lying between Manoa and Palolo Valleys, is poorly connected to Area 2 and is down gradient from it; therefore it is excluded from the main Southern Oahu basal aquifer. Area 5, to the east of Diamond Head, is even more poorly connected to Area 1. Another important subdivision is the westernmost sector, which is composed of Waianae basalt rather than the Koolau basalt that constitutes the remainder of Southern Oahu. A moderate erosional unconformity at the top of the Waianae volcanic series is covered by the younger Koolau rocks along a northerly extension from about Oneula Beach on the Ewa coast to the Wahiawa high level aquifer in the vicinity of Wheeler Field. Groundwater passes from the Koolau aquifer through the unconformity into the Waianae aquifer.

Volume Computed from Geometry and Effective Porosity of the Basal Aquifer

In computing initial storage, the basal lens is assumed to exactly conform to the Ghyben-Herzberg relationship and to have a sharp lower interface. From Darcy's Law combined with the Ghyben-Herzberg principle the shape of both the upper and lower surfaces of a free, unconfined lens is derived as a parabola. Volumes are easily obtained from the equation of the parabolas so long as the Darcy parameters are known. In this sense the volume is a function several variables and constants, as follows:

(1) $V = f(h_o, x, q, y, k)$.

In the above, V is volume, h_o is initial head, x is distance upgradient from $x = o$, q is specific flux multiplied by the depth of flow, y is the width of aquifer section, and k is hydraulic conductivity. Values of h_o, x, and y can be measured relatively accurately, but the reliability of estimates for q and k depends on the precision of hydrologic budgeting and the analysis of pumping tests.

A more straightforward computation that eliminates the need for q and k, but on the other hand requires knowledge of the easily measured hydraulic gradient, involves parabolic equations expressed in terms of h and x only. The form of the parabolic equation for the upper free surface of the lens is, as defined in figure 1:

(2) $h = h_o + bx^{1/2}$

The value of b is readily obtained if the regional hydraulic gradient between h at some distance x and h_o at $x = o$ is known. For instance, if the gradient is 1 ft/mile, as is typical in Southern Oahu inland of the Pearl Harbor Springs, the value of b would be:

(3) $b = \dfrac{h - h_o}{x^{1/2}} = .0138$

Equation (2) would then become,

(4) $h = h_o + .0138 \times ^{1/2}$.

The volume, A, of a one foot wide strip between the upper and lower parabolas is obtained by solving,

(5) $A = 41 \int_o^x h dx$

which yields,

(6) $A = 41 (h_o x + 2/3\, bx^{3/2})$.

The total water volume, V, is calculated by multiplying the above by the width of the aquifer, y, and the effective porosity, S,

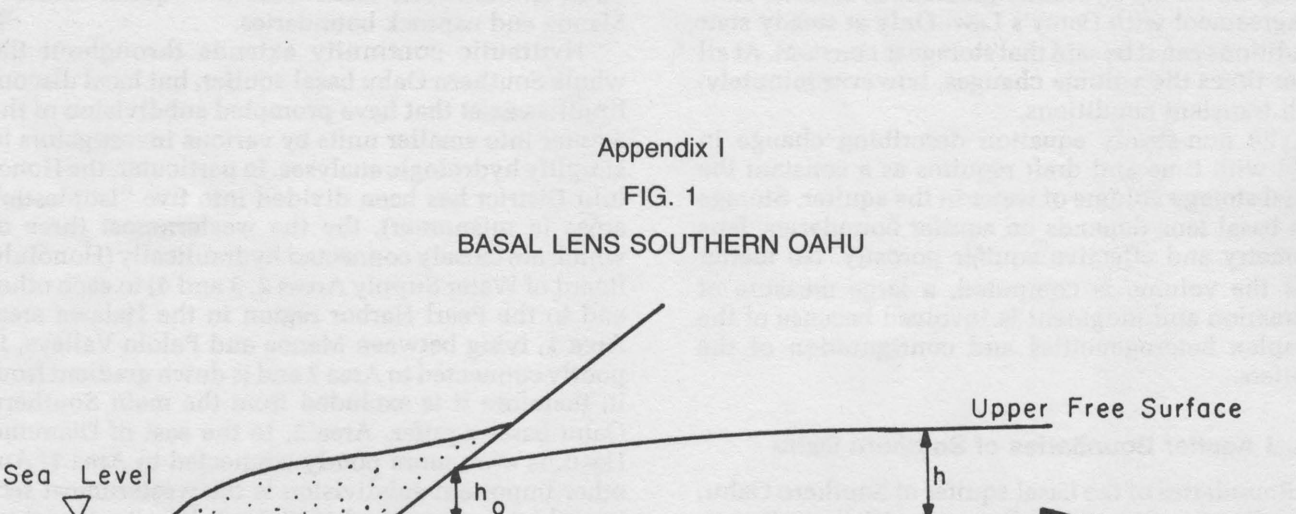
(7) $V = 41yS \left[h_o x + 2/3\, bx^{3/2}\right]$

Volumes of subdivisions of the basal aquifer have been computed using this formula and compared with calculations utilizing a single average lens

Appendix I

FIG. 1

BASAL LENS SOUTHERN OAHU



thickness. The difference between the two methods is very small for Southern Oahu because the hydraulic gradient is so slight. For the unconfined portions of the aquifer the simpler method is sufficiently accurate. The volume of the triangular segment of the aquifer confined beneath the caprock must be added to the unconfined volume to give total storage. This smaller volume accounts for less than one tenth of total storage.

Effective porosity is commonly assumed to be ten percent in the basaltic aquifers of Hawaii. This approximation is by and large a matter of judgment, a convenient decimal expressing a global parameter that is incapable of finer resolution. Relatively little attention has been devoted to seeking local and regional values of porosity because the heterogenity of basaltic aquifers is so overwhelming. C. K. Wentworth (1951) determined from laboratory measurements a range of porosity of 5.2 to 51.4 percent in drill cores of Koolau basalt. Williams and Soroos (1973) analyzed the results of many pumping tests and computed unconfined specific yields of .0004 to greater than unity, an impossible value. Total porosities have been determined by gravimetric surveys (Huber, et al, 1971), from which an average porosity of 26.4 percent was found for Koolau basalt in the vicinity of Schofield Shaft, the U.S. Army pumping station in the Wahiawa high level aquifer. Local values of porosity manifestly vary greatly from place to place.

Utilizing an effective porosity of 10 percent in equation (7) and for the lens segment beneath the caprock, the initial storage of the aquifer of Southern Oahu and its subdivisions were computed to be as follows:

Subdivision	Initial Volume, V_0, as a Range (ft.^{3} x 10^9)	Initial Volume, as a Probable Value (ft.^{3} x 10^9)
Honolulu Areas 2, 3, 4	85 to 110	91
Pearl Harbor Koolau Basalt	—	280
Pearl Harbor Waianae basalt	—	70
Pearl Harbor, Total	300 to 460	350
Honolulu plus Pearl Harbor, Total	385 to 570	440

In the transient formulation of the model, $h = f(t,D)$, and when draft, D, is greater than recharge, I, the effect of V_0 diminishes with time until ultimately it is consumed. For $I>D$, at the steady state, V_0 no longer appears in the formulation.

APPENDIX II

Transmissivity and Groundwater Flow

The essential aquifer parameter required to compute aquifer flux and to predict aquifer behavior in response to pumping is called transmissivity, which is defined as the volume of water flowing per unit time through a unit width of aquifer throughout the depth of flow under unit hydraulic gradient. Hydraulic conductivity is transmissivity divided by depth of flow. In well hydraulics, drawdown is predominantly inversely proportional to transmissivity, as shown by the basic relationship for confined aquifers (or unconfined aquifers in which depth of flow is much greater than drawdown, which is characteristic of the basal aquifers of Southern Oahu):

(1) $s = \frac{Q}{4\pi T}\, w(u)$

in which s is drawdown, Q is constant draft, T is transmissivity, and w(u) is a function of the equality, $u = r^2S/4Tt$, wherein r is radial distance from the pumping source, S is coefficient of storage or specific yield, and t is time. For computations of aquifer flow the relationship:

(2) $Q = TiL$

pertains, in which i is hydraulic gradient and L is width of the outflow section. Equation (2) is an extension of Darcy's Law.

Ordinarily, T is calculated from data obtained in controlled pump tests, but well hydraulic formulas are applicable only under a narrow set of aquifer and pumping conditions. No pumping tests conducted in the basal aquifers of Southern Oahu have met such conditions; wells partially rather than fully penetrate the aquifers, the lower limit of flow in the aquifers is not fixed, and the aquifers are laterally bounded rather than infinite or even extensive. Nevertheless, pumping tests where judiciously interpreted have provided reasonable estimates of transmissivity and hydraulic conductivity. It is important to recognize that all values of these parameters and of specific yield obtained from pumping tests in Hawaii, especially in basal aquifers, are approximations generated from a combination of mathematical analysis and personal judgment.

C. K. Wentworth (1938) was the first to plan and carry out a controlled pumping test in Hawaii. The Waialae Pumping Station (Shaft 7) was pumped for a long period at a constant rate and drawdowns were measured in numerous observation holes at different distances from the shaft. Wentworth, using the equivalent of the steady state formulation, computed hydraulic conductivities of 1,818 to 3,516 ft/day, convertible in his formulation, in which the depth of flow was 400 feet, to transmissivities of 727,000 to 1,406,400 ft^2/day. Williams and Soroos (1973) summarized the data for all major pumping tests done after Wentworth's effort and recomputed values of transmissivity, hydraulic conductivity and specific yield. Although some of their results are invalid, in particular the cases where they assumed an impermeable bottom for basal aquifers, by and large the range of computed transmissivities is similar to that of Wentworth's. For the ten best tests in the unconfined basal aquifer of the Pearl Harbor region the average computed hydraulic conductivity, assuming that the full depth of fresh water flow in the aquifer was responding to the pumping, was 1,841 ft/day. This is probably a low value because wells are partially penetrating and the full depth of flow is not likely generated.

In heterogeneous, non-isotropic aquifers, such as the basal lens of Southern Oahu, pumping tests are largely influenced by local conditions and may not reflect the global characteristics of the aquifer. One way to obtain these large scale characteristics is by analyzing head changes over the entire area of the aquifer in response to the equivalent of controlled total aquifer pumping, either from the instantaneous start of all pumps or their instantaneous shutdown. On a few occasions these pumping conditions have taken place in the Pearl Harbor region when irrigation for sugar was suddenly terminated by industry wide strikes, followed by resumption of pumping after the strike was settled. The best data set was collected for the 1958 strike, which commenced on February 14 and continued to the end of May. All plantation pumps were turned off nearly simultaneously on February 13-14, providing nicely defined initial conditions for analysis of head recovery. The start of pumping in June was erratic and data was not as carefully and comprehensively collected as for the recovery period.

Visher and Mink (1960) briefly discuss the hydrologic conditions of the 1958 strike and computed a value for transmissivity based on drawdowns that occurred upon the resumption of pumping. The method of analysis was not explained nor were the assumptions outlined. The very low transmissivity computed, 0.59×10^6 ft²/day, suggests that the analysis was faulty and that the drawdown data, taken over only seven days, was poor.

On the other hand, recovery for the period February 14 through May 31 was carefully measured at the end of each month at 24 non-pumping wells and observation holes. At Well 250-2, near the Wahiawa high level boundary of the basal aquifer, readings were taken on an approximately weekly basis. The entire data set exhibits the recovery trend. From detailed analysis of recovery at six of the sites the global parameters of the Pearl Harbor basal aquifer have been estimated.

Analysis of Head Recovery, 1958 Strike

Prior to the strike the average draft in the Pearl Harbor region was approximately 160 mgd, which was nearly instantaneously reduced to 40 mgd on February 14. In 1958 the Board of Water Supply had not yet established large pumping stations west of Halawa Shaft. The 40 mgd that continued to be pumped was shared by the U.S. Navy (about 20 mgd from Waiawa Shaft), small Board of Water Supply stations (Pearl City, Aiea), minimal plantation draft, and a few private wells. By assuming the steady state condition had been achieved for a rate of 160 mgd prior to the strike, the recovery following reduction to 40 mgd is mathematically equivalent to drawdown induced by a pumping rate of 120 mgd. The assumption is reasonable; it is made for every pumping test in aquifers that are being exploited.

The recovery data analysis would be unambiguously simple if all pumping were from a single site as is required by equation (1), but in the Pearl Harbor region the equivalent pumping for the equivalent drawdown (actual recovery) analysis was distributed among five major pumping centers — Halawa, Waimalu, Waiawa, Waipahu and Ewa. Before the strike the long term distribution of total plantation draft among these centers was as follows: Halawa, 17 percent; Waimalu, 13 percent; Waiawa, 7 percent; Waipahu, 20 percent; and Ewa, 43 percent. For the analysis the equivalent pumping rate of 120 mgd is divided among the pumping centers by the above proportions.

The solution of equation (1) for the recovery data set, in which only the composite equivalent drawdown is given at each site, can't be solved unless a single distance between the pumping region and the observation site is assumed. The artifice of employing the geometric mean of the distances from a site to each pumping center as the single value for distance was selected because drawdown varies with the logarithm of distance. Only those observation sites were considered for which a sensitivity test indicated that the value of drawdown computed for a single value of distance, taken as the geometric mean of all distances, differed by 25 percent or less from the sum of drawdowns obtained by solving equation (1) for each real distance to a pumping center. The sensitivity analysis was made by solving equation (1) for the assumed values $t = 50$ days, $S = .10$, $T = 1 \times 10^6$ ft²/day, and for the values of draft apportioned to each center as noted earlier. These values assigned to T and S are estimates of the order of magnitude known for the Pearl Harbor region; t is the mid period of the recovery data set. Table 1 summarizes calculations for the observation sites meeting the test.

Appendix II
TABLE 1

Sensitivity analysis for determining acceptability of the geometric mean of distance from pumping centers to observation sites.

Wells; distance (r) in ft.x1000; drawdown(s) in ft.

Pumping Center	Pumpage mgd	239-1 r	239-1 s	250-2 r	250-2 s	330-5 r	330-5 s	196-1B r	196-1B s	T-46 r	T-46 s	T-47 r	T-47 s
Halawa	22	20	.28	45	.05	55	.02	17	.34	37	.09	34	.11
Waimalu	16	15	.27	40	.05	40	.05	7	.53	27	.13	25	.15
Waiawa	8	20	.11	28	.06	32	.05	10	.61	19	.12	13	.17
Waipahu	24	30	.16	30	.16	27	.19	25	.22	24	.24	15	.41
Ewa	52	45	.12	40	.17	32	.30	30	.35	35	.24	24	.52
Total	122		.94		.49		.61		2.05		.82		1.36
Geometric mean (r) and related (s)		24	1.18	36	.52	36	.52	16	2.11	28	.94	21	1.46
Percent diff. (s)			25.5		6.1		14.8		2.9		14.6		7.4

Global values for the parameters T and S were computed for the recovery (equivalent drawdown) set for the six observation sites listed in Table 1. Recoveries at the 14 day, 45 day and 75 day intervals in conjunction with the geometric mean distances were employed along with total equivalent draft of 16×10^6 ft³/day in equation (1) and in the Jacob semi-log plot method. Equation (1) was solved iteratively for T and S. In the Jacob method these parameters are determined graphically.

Table 2 summarizes results of the calculations.

The mean T is 1.5×10^6 ft²/day and the mean S is .034 as computed by equation (1). For the Jacob method, respective means are 1.7×10^6 ft²/day and .063. During the strike interval the average head was about 25 feet, from which the global hydraulic conductivity could be roughly estimated by the relationship:

$$(3) \quad k = \frac{T}{b} = \frac{(1.6 \times 10^6)}{(25 \times 41)} = 1561 \text{ ft/day}$$

The above parameters are concordant with the values obtained by Wentworth (1938) and those computed by Williams and Soroos (1973) but more accurately reflect regional conditions.

Appendix II
TABLE 2

Global values of T and S, Pearl Harbor basal aquifer.
Values from Theis equation and Jacob semi-log plot.
1958 Strike Recovery Data, Feb. 14 - May 31.

Site	Dist. (r) geom. mean ft.x1000	Observed Drawdown (s), ft. time (t) 14 days	45 days	75 days	Theis T (ft^2x10^6)	S	Jacob T (ft^2x10^6)	S
239-1	24	.75	1.74	2.01	1.55	.041	1.48	.08
250-2	36	.56	1.68	2.13	1.06	.026	1.64	.02
330-5	36	.93	2.37	2.59	1.21	.014	2.58	.03
196-1B	16	1.04	1.84	1.86	2.67	.028	1.59	.14
T-46	28	.34	1.45	1.90	.93	.059	1.40	.08
T-47	21	1.08	2.28	2.58	1.34	.036	1.48	.03
Average					1.5	.034	1.7	.063
Std. Dev.					.63	.015	.44	.050

Computation of Flux by Q = TiL

For laminar flow, the foundation of Darcy's Law, total groundwater flux is given by the simple expression, Q=TiL, in which i is the hydraulic gradient, or head loss per unit horizontal distance in the direction of flow, and L is section width. The hydraulic gradient in the undisturbed sector of the Pearl Harbor basin, one mile or more upgradient of the springs and the major pumping stations, is about one foot per mile. During the 1958 strike the gradient in the four mile distance down gradient of Well 250-2 and upgradient of T-29 averaged one foot per mile. This is also the gradient in the Waianae aquifer and in the Honolulu aquifer.

The distance along an equipotential in the central Pearl Harbor region from the Waianae crest to Moanalua Valley is 16 miles. Utilizing this distance for L, the gradient of one foot per mile for i, and transmissivity, T, of 12 mgd/ft. (1.6×10^6 ft^2/day), the total flux moving through the Pearl Harbor basal aquifer is computed as 192 mgd. This value is somewhat lower than but consistent with values derived from hydrologic budgeting. Although conservative, the value clearly illustrates that no matter how the natural flux is computed it is on the order of 200 mgd for the Pearl Harbor region. Return irrigation from the Waiahole system expands it to about 215 mgd.

Total natural movement to Pearl Harbor includes a component that flows toward the springs from the Honolulu District. A rough estimate of total groundwater flow that includes this component and a value of L of 20 miles rather than 16 miles yields a total natural flux of 240 mgd, also somewhat below hydrologic budget estimates by about ten percent. Under the development conditions now prevailing in Honolulu, the natural flow toward the springs is greatly reduced.

APPENDIX III

Derivation of the Robust Analytical Model

In porous media the governing equation of flow that combines the continuity relationship with Darcy's Law is expressed as the following partial differential equation (for simplicity the equation is written in one dimension with constant hydraulic conductivity, k):

$$(1) \quad \frac{d\left[kh\,\frac{dh}{dx}\right]}{dx} = S\,\frac{dh}{dt} + W(x,\ t)$$

in which h is head of the water table above a prescribed datum, x is distance along a streamline, k is hydraulic conductivity, S is specific yield, t is time, and W is a source-sink term. The term on the left contains the Darcy substitution for the mass balance relationship, which is

$$(2) \quad \frac{dq}{dx} = S\,\frac{dh}{dt} + W(x,\ t)$$

in which q is specific flux.

In the above equation assume that all extractions, D, and sources, I, are independent of the x coordinate and operate such that the resultant groundwater flow is uniform throughout the extent of the aquifer, and further assume that ultimate leakage, L, at the coastline is a function of head. The source-sink term in equation (2) would then be expressed as W[I, D, L(h)]. The assumptions for D and I are not unreasonable for extensive basal aquifers of Hawaii in which transmissivity is very high and therefore the radius of influence of an extraction point is wide but shallow, and the major portion of recharge enters the aquifer from the wet mountain area. Leakage is known to be a function of head as explained below. Figure 1 illustrates the elements of the model for a basal lens.

The steady-state balance equation, given I>D, is

$$(3) \quad Q = I - D = L$$

in which Q is total flow. Steady flow in a basal lens in conformance with Darcy's Law is

$$(4) \quad Q = ky\ (B + 1)\ h\,\frac{dh}{dx}$$

in which y is the constant width of the aquifer and $B = g_f/g_s - g_f$, wherein g_f is the density of fresh water and g_s the density of salt water, so that in the normal ocean-fresh water system, B is approximately 40. In all ensuing equations (B + 1) will be replaced by 41, the constant employed commonly in Hawaii. Thus,

$$(5) \quad Q = 41kyh\,\frac{dh}{dx}$$

for which, assuming discharge along a line at the sea coast (h = 0, x = 0), the solution is

$$(6) \quad Q = \frac{41ky}{2x}\,h^2$$

Under steady-state conditions the assumption of a line discharge makes no material difference in computed leakage at any vertical section along the parabolic extent of the lens. In equation (6) Q is constant and is fixed by the relationship between x and h. At any given distance from the coast the equation may be written as:

$$(7) \quad Q = ch^2$$

for which c = 41ky/2x = constant.

Given the assumptions shown in Figure 1, equation (7) may also be expressed

$$(8) \quad L = ch^2$$

At initial conditions when D = 0,

$$(9) \quad I = Q_o = L_o$$

and therefore the constant c at a selected section would be

$$(10) \quad c = \frac{I}{h_o^2}$$

This system relationship is necessary for the solution of the balance equations.

Returning to equation (2) and assuming no change in specific flux in the x direction, the left-hand side of the equation becomes $dq/dx = 0$, and the whole equation becomes:

$$(11) \quad S \frac{dh}{dt} + W[I, D, L(h)] = 0$$

What the above says is that at any given time a steady flow exists, so that the solution of $h = f(t)$ applies to a succession of steady-states at a particular location, e.g., an observation well.

Equation (11), written in total rather than specific terms, is:

$$(12) \quad 41SA \frac{dh}{dt} = I - D - ch^2$$

in which A is the lateral area of the aquifer. Let $b = 41SA$, then,

$$(13) \quad \int \frac{dh}{I - D - ch^2} = \frac{1}{b}\int dt$$

Taking as the average initial thickness of the lens $Z_o = 41h_o$, and from the volume calculation $bh_o = V_o$, the value of the system constant is, $b = V_o/h_o$.

Written in terms of changes over a specified time interval with the initial conditions taken at the start of the interval, equation (13) is written as:

$$(14) \quad \int_{h_i}^{h_{i+1}} \frac{dh}{I - D - ch^2} = \frac{1}{b}\int_{t_i}^{t_{i+1}} dt$$

This equation may be employed to determine the head change from h_i at the start of the interval t_i to h_{i+1} for an average value of D in the interval. In effect, D is allowed to vary from one interval to another though it is constant within the interval. The value of I is the same for all intervals, and therefore the constant $c = I/h_o^2$ does not change from one interval to another. The constant b depends on V and h at the start of each interval, but because the proportion $V_o/h_o = V_i/h_i = V_{i+1}/h_{i+1}$, the value of b is the same for every interval and is employed as V_o/h_o, obtained from the original initial condition.

The solution of equation (14) for I>D is as follows:

$$(15) \quad h_{i+1} = h_o\left[\frac{I-D}{I}\right]^{\frac{1}{2}} \left\{ \frac{\left((I-D)^{\frac{1}{2}} + \frac{h_i}{h_o}(I)^{\frac{1}{2}}\right)\exp\left(\frac{2[(I-D)I]^{\frac{1}{2}}(t_{i+1} - t_i)}{V_o}\right) - (I-D)^{\frac{1}{2}} + \frac{h_i}{h_o}(I)^{\frac{1}{2}}}{\left((I-D)^{\frac{1}{2}} + \frac{h_i}{h_o}(I)^{\frac{1}{2}}\right)\exp\left(\frac{2[(I-D)I]^{\frac{1}{2}}(t_{i+1} - t_i)}{V_o}\right) + (I-D)^{\frac{1}{2}} - \frac{h_i}{h_o}(I)^{\frac{1}{2}}} \right\}$$

In the above, the variables subscripted with i refer to values at the start of an interval and those with i+1 at the end of an interval, and the term $t_{i+1} - t_i$ is the length of the interval. The equation is in a form that readily utilizes the draft data collected in Hawaii for many years, particularly for Southern Oahu where average monthly and annual draft have been reliably reported since 1910. Solution of the equation gives storage head of the system at the end of each interval.

The heads h_i and h_{i+1} are not steady-state heads. To obtain the equilibrium head for a set of conditions prevailing in an interval, let t go to infinity and equation (15) becomes:

$$(16) \quad h = h_o\left[\frac{I - D}{I}\right]^{\frac{1}{2}}$$

A steady state is impossible for $I \leq D$.

Equation (15) provides the storage head (h_v), that is, the parameter reflecting the vertical dimension of volume of fresh water for the given time. It differs from the operating head (h_p), which is the head condition induced by pumping. The difference between storage and operating heads is restricted to the top of the basal lens; when pumping ceases entirely, the head recovers to approach the true volumetric head. Indeed, the disparity between storage and operating head, in conjunction with the long transient stage of head decline, in large measure incorporates Wentworth's concept of bottom storage.

Although hydraulic conductivity in the basalt aquifers is extremely high, on an order greater than 1,000 ft./day, and drawdown cones due to pumping are shallow and radially extensive, operating heads are significantly lower than storage heads because all Hawaii aquifers are effectively bounded. For example, the major basal aquifer of Southern Oahu has as its boundaries the Manoa Valley fill, the high level Schofield Aquifer, the rift zones of the Koolau and Waianae ranges, and the caprock wedge on the coastal plain. The effect of these boundaries is to superimpose additional drawdown on the primary shallow drawdown cone, leading to an aquifer-wide operating head substantially lower than the true storage head given by equation (15)

Equation (14) may be solved for conditions of O<I<D and I = D. For O<I<D, the solution is

$$(17) \quad h_{i+1} = h_o\left[\frac{D - I}{I}\right]^{\frac{1}{2}} \tan\left(\arctan \frac{h_i}{h_o}\left[\frac{I}{D - I}\right]^{\frac{1}{2}} - \frac{[(D - I)I]^{\frac{1}{2}}}{V_o}(t_{i+1} - t_i)\right)$$

When I = D, the solution is

$$(18) \quad h_{i+1} = \frac{h_i h_o V_o}{h_o V_o + h_i I(t_{i+1} - t_i)}$$

In the above, for $h_i = h_o$:

$$(19) \quad h = \frac{h_o V_o}{V_o + I_t}$$

This expression asserts that the sustainable yield of an aquifer can never be equal to total recharge because as t increases, h goes asymptotically to zero and never attains a steady state. Only when I>D can a sustainable yield be assigned to an aquifer.

Equations 15 through 19 are used to simulate and predict heads. In the equations D is measured, h_o is reliably known, I has been determined by hydrologic budgeting and flow analysis, and V_o has been estimated by assuming an aquifer porosity of 10 percent and specified subsurface boundaries. The least reliable assumption is the estimate of porosity. To test the effect of porosity on computed head, a sensitivity analysis was made for a range of porosities from one to 30 percent for the simulation period 1880 to 1980. Table 1 lists the results.

The difference in head over the century of simulation between one percent porosity and 30 percent porosity is 8.8 feet, which is substantial but relatively small considering the thirty fold difference in initial storage. Between the one percent and 10 percent porosities the head difference is 4.6 feet, and between 10 percent and 30 percent it is 4.2 feet. Aquifer porosity is not at all likely to be as high as 30 percent or as low as one percent. In the range where it probably falls — five percent to 20 percent — the maximum computed head difference is four feet for 100 years of simulation. Evidently head is not profoundly influenced over a reasonable range of assigned porosities.

Table 1 continues the sensitivity analysis to the end of the century based on a steady net draft of 225 mgd starting in 1980 for Southern Oahu. Over the twenty year period the 30 percent porosity head ends up 4.8 feet higher than the one percent porosity head. For 10 percent porosity the head would be 1.6 feet lower than the 30 percent porosity head and 3.2 feet higher than the one percent porosity head. The final equilibrium head, h_e, would be 15 feet no matter what the porosity since V_o does not appear in the steady state equation (see equation (16)).

Another type of analysis is displayed in Figure 2. Here two different values of I for the Koolau portion of the Pearl Harbor region are coupled with different values of V_o, and the simulations are matched against the head record from 1916 to 1978. The poorest fits are for minimum I (170 mgd) and minimum V_o (150×10^9 ft.^{3}), and maximum I (250 mgd) and maximum V_o (560×10^9 ft.^{3}). Better fits are obtained for I = 250 mgd, $V_o = 280 \times 10^9$ ft.^{3}, and I = 170 mgd, $V_o = 280 \times 10^9$ ft.^{3}. By computing curves using combinations of reasonable values of I and D, a best simulation is obtained, which in turn determines acceptable values for these components. In fact, the hydrologic budget value for I and V_o based on porosity of 10 percent provides the best simulation.

Appendix III
TABLE 1

Sensitivity Analysis

Effect of different values of porosity and initial volume of storage head

Region: Manoa Valley to Waianae Crest

Conditions (draft in mgd;
volume in $ft^3 \times 10^9$;
head in ft.)

D = average net draft
I = 281 mgd
Δt = 1,825 days (5 yrs.)
h_o (Well 244) = 33.5 ft.
V_o = variable

		h				
		Porosity and Volume				
Period	D	.01 44	.05 220	.10 440	.20 880	.30 1320
1880-85	18	32.5	33.0	33.2	33.4	33.4
1886-90	24	32.1	32.3	32.9	33.2	33.3
1891-95	62	29.7	31.3	32.1	32.7	32.9
1896-1900	103	26.9	29.4	30.8	31.9	32.4
1901-05	119	25.6	27.8	29.6	31.1	31.8
1906-10	147	23.4	26.1	28.2	30.2	31.1
1911-15	158	22.3	24.7	27.0	29.3	30.4
1916-20	142	23.5	24.3	26.3	28.6	29.8
1921-25	165	21.9	23.8	25.3	27.9	29.2
1926-30	157	22.3	23.0	24.7	27.2	28.7
1931-35	147	23.1	23.1	24.4	26.8	28.3
1936-40	136	24.0	23.4	24.3	26.5	27.9
1941-45	185	20.3	22.2	23.5	25.8	27.4
1946-50	145	23.0	22.6	23.4	25.5	27.1
1951-55	153	22.7	22.6	23.3	25.2	26.7
1956-60	153	22.7	22.6	23.2	25.0	26.4
1961-65	173	21.1	22.0	22.7	24.5	26.0
1966-70	195	18.9	20.9	22.0	24.0	25.5
1971-75	210	17.2	19.7	21.2	23.3	25.0
1976-80	225	15.6	18.5	20.2	22.6	24.4

Conditions: same, except
that h_i (244) = 21.0 ft.
for all values of V_o

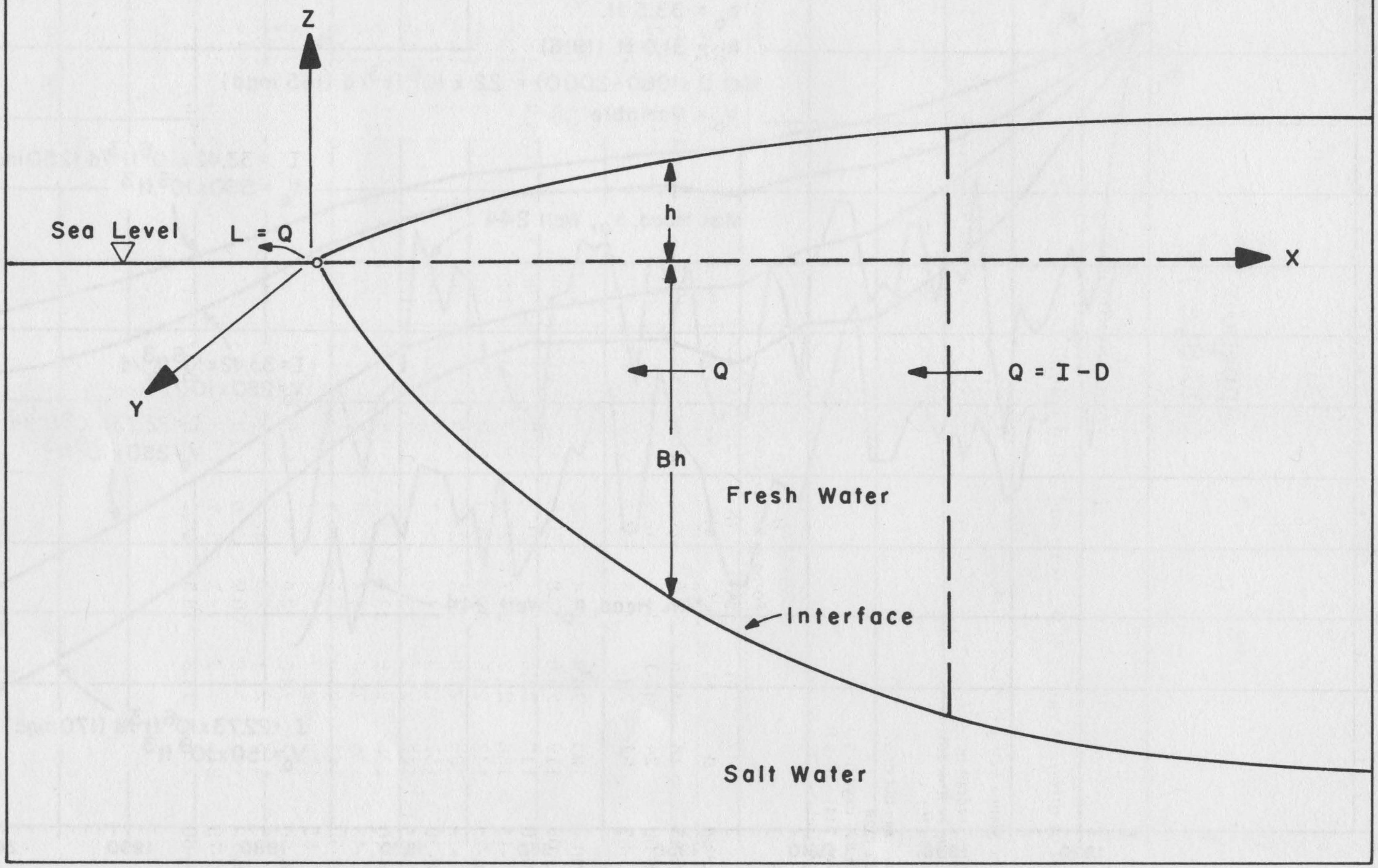
		h				
		Porosity and Volume				
Period	D	.01 44	.05 220	.10 440	.20 880	.30 1320
1980	225	19.3	20.6	20.8	20.9	20.9
1981	"	18.2	20.3	20.6	20.8	20.9
1982	"	17.3	19.9	20.4	20.7	20.8
1983	"	16.7	19.6	20.3	20.6	20.7
1984	"	16.3	19.3	20.1	20.5	20.7
1985	"	15.9	19.1	19.9	20.4	20.6
1986	"	15.7	18.8	19.8	20.3	20.6
1987	"	15.5	18.6	19.6	20.3	20.5
1988	"	15.4	18.4	19.5	20.2	20.4
1989	"	15.3	18.2	19.3	20.1	20.4
1990	"	15.2	18.0	19.2	20.0	20.3
1991	"	15.1	17.8	19.1	19.9	20.3
1992	"	15.1	17.6	19.0	19.8	20.2
1993	"	15.1	17.5	18.8	19.8	20.1
1994	"	15.0	17.3	18.7	19.7	20.1
1995	"	15.0	17.2	18.6	19.6	20.0
1996	"	15.0	17.1	18.5	19.5	20.0
1997	"	15.0	16.9	18.4	19.5	19.9
1998	"	15.0	16.8	18.3	19.4	19.9
1999	"	15.0	16.7	18.2	19.3	19.8

Equilibrium head (h_e) = 15.0

Appendix III

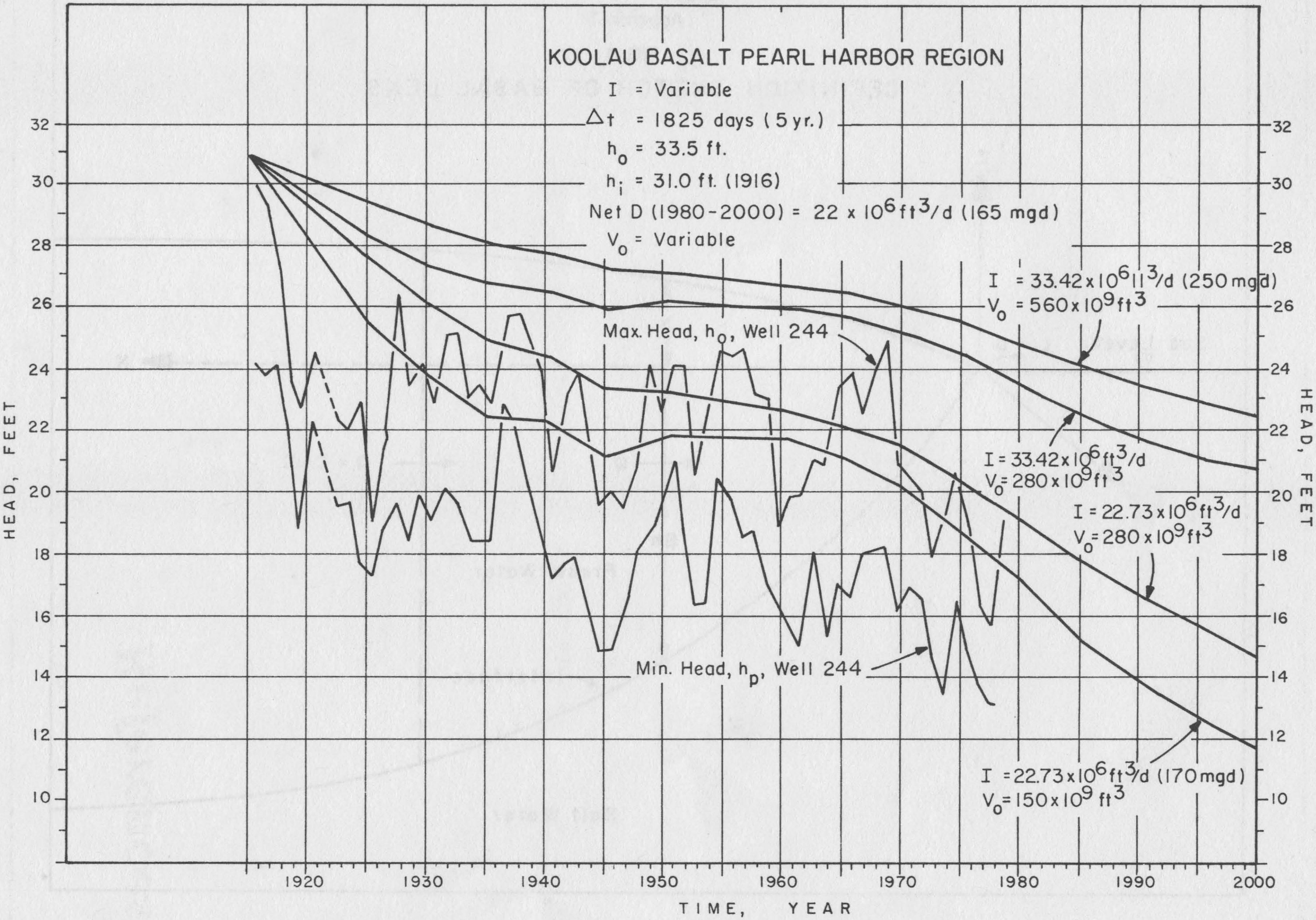
FIG. 1

DEFINITION SKETCH OF BASAL LENS



Appendix III

FIG. 2



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